

A THESIS

on

DESIGN AND ECONOMICAL COMPARISON

REINFORCED CONCRETE

ENCASED STEEL BEAM

STRUCTURAL STEEL TRUSS

THROUGH PLATE GIRDER

TYPES OF BRIDGES

PRESENTED TO THE FACULTY

GEORGIA SCHOOL OF TECHNOLOGY

May 15, 1933

As part fulfillment of the requirements

for the Degree

MASTER OF SCIENCE IN CIVIL ENGINEERING

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BACHELOR OF SCIENCE IN CIVIL ENGINEERING

CLEMSON A and M COLLEGE of SOUTH CAROLINA

June 5, 1930

Approved by:

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INTRODUCTION

The purpose of this Thesis is to design and economically compare four types of highway bridges now in common use, namely, the reinforced concrete type, the encased steel beam type, the through plate girder type, and the through Pratt truss type of bridge. The span for this investigation shall be 128 ft. divided as follows: 4 spans at 32 ft. for the reinforced concrete, 2 spans at 64 ft. for the encased beam type, 2 spans at 64 ft. for the plate girder, and a single span of 128 ft. for the truss bridge. Width of roadway for each bridge shall be about 40 ft., and each shall be designed for a 12 ton live loading. Complete description and design data for each bridge is given before the design of that bridge is started.

These bridges are to be compared from the standpoint of first cost only, and under the assumption that local conditions for steel and concrete structures are identical. For type selection of any structure these and other conditions such as maintenance, life, location, etc. would be taken into consideration.

References were made throughout the design and preparation of this Thesis to the following books and notes: "Design of Highway Bridges", by Kirkham, "Highway Bridges", by Ketchum, "Handbook of Cost Data", by Gillette, "Concrete Design Notes", by Snow, "American Civil Engineers Handbook", by Merrimen and Wiggam. Each bridge will be designed and a cost sheet compiled for that particular bridge. In the conclusion a comparison of the cost of the four bridges will be made.

d-Wind Stresses pp 60 to pp 63

e-Design of Sections pp 63 to pp 65

f-Design of Joints pp 65 to pp 68

g-End Plate Design pp 68 to pp 69

h-Cost pp 69 to pp 70

6-CONCLUSION pp 71

7-DESIGN TABLES

REINFORCED CONCRETE SLAB, BEAM, GIRDER,

COLUMN TYPE BRIDGE

Span - 4 at 32 ft. - 128 ft.

DESCRIPTION OF BRIDGE

General Description- The first bridge to be designed shall be the reinforced concrete slab, beam, girder, column type, made up of four spans at 32 ft., giving an overall length of 128 ft. The roadway shall be 40 ft. (c to c of railing). The floor shall be reinforced concrete resting on floorbeams spaced at 5 ft. and running lengthwise the bridge. These beams shall rest on girders spaced at 32 ft. (one at each end of each section). Columns shall be spaced at 10 ft. giving a cantilever overhang of 5 ft. of girder at each end. Wearing surface of 25# per. ft. of roadway to be used.

LOADS- Dead Load- The dead load shall consist of the weight of the floor slab, beams, girders, columns and wearing surface. Reinforced concrete shall be assumed to weigh 150# per. cu. ft.

Live Load- The bridge will be designed for a 12 ton truck live loading. 19,200# on two rear wheels (9,600# each wheel) and 4,800# (2,400# each) on front wheels.

Impact- 30% shall be added for impact.

Wind Load- No appreciable effect on this type bridge.

General Dimensions-

Span- 4 spans at 32 ft. out to out = 128 ft. overall.

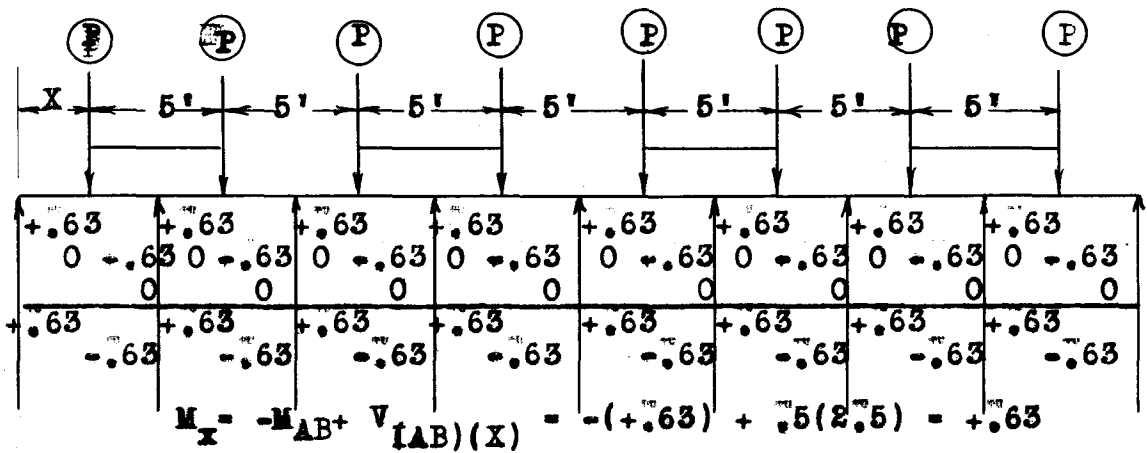
Width of roadway- About 38 ft.

Spacing of beams- 5 ft. c to c lengthwise the bridge.

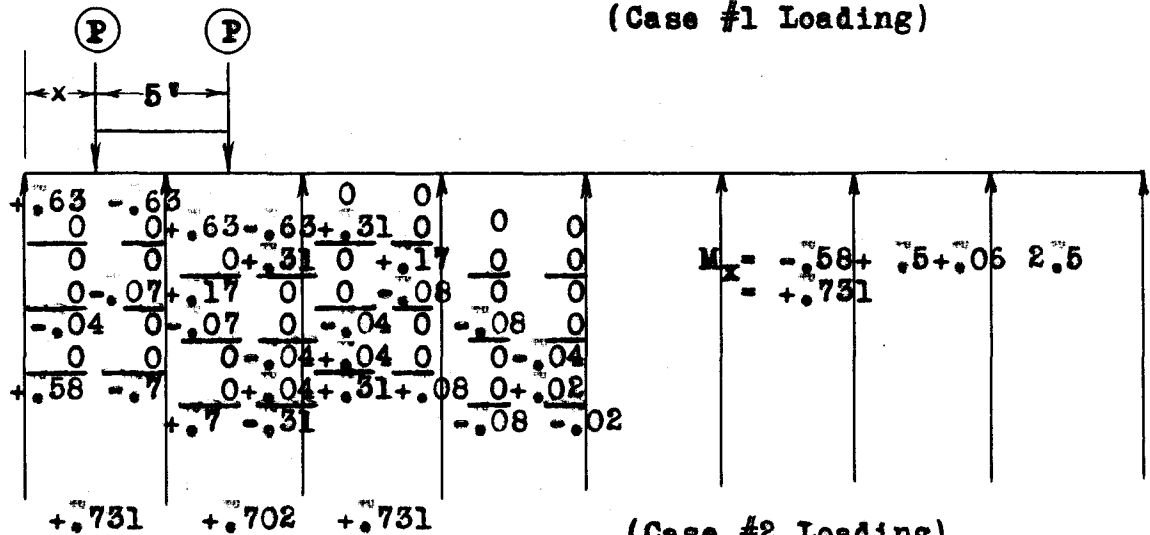
Spacing of girders- One girder at each end of each section

Spacing of columns- 10 ft. c to c with a cantilever overhang of girder of 5 ft. on each side.

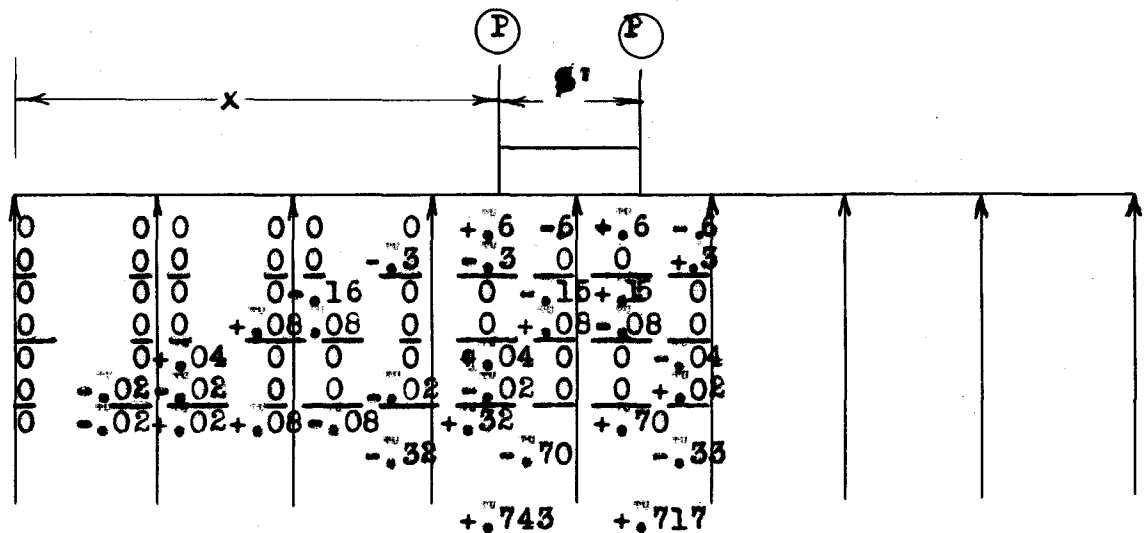
MOMENTS ON SLAB : HARDY CROSS METHOD
(Concentrated Live Loads)



(Case #1 Loading)



(Case #2 Loading)



(Case #3 Loading)

M (For each loading) = $(1/8)wl = (1/8)P(8) = .625 (P - 1)$
 M given at left and right of each column and at the center between columns.

--- UNIFORM DEAD LOAD = 1# Per Ft. ---

Moment over supports = $(1/12)w_1^2 = (1/12)1(5)^2 = 2.08$
 Moment at c. l. = $(1/24)w_1^2 = (1/24)1(5)^2 = 1.04$

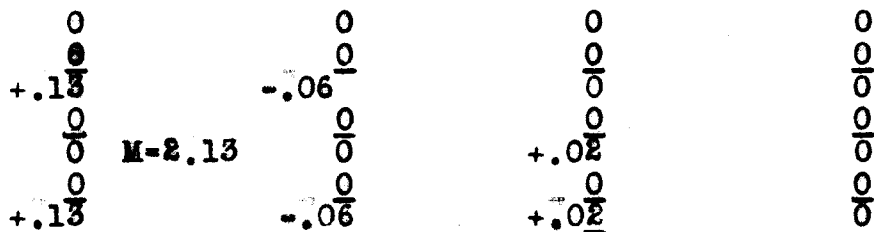
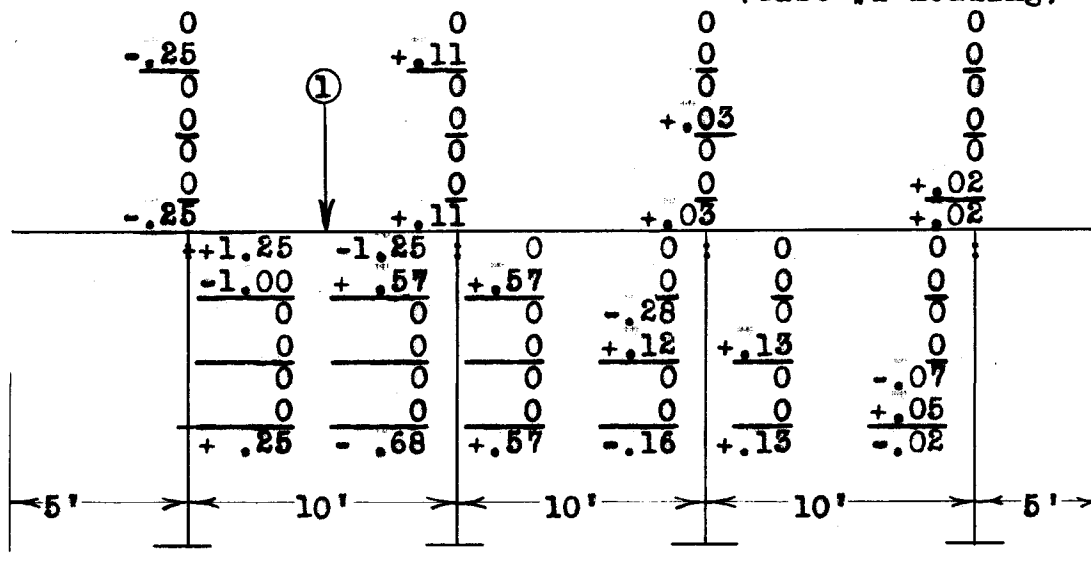
MOMENTS ON GIRDERS : HARDY CROSS METHOD (Concentrated Live Loads)

Assume beam to be 2' by 2.5', $EI/L = 1/12(2)(2.5)^3/10$
 $= .260$ and $k = 4$

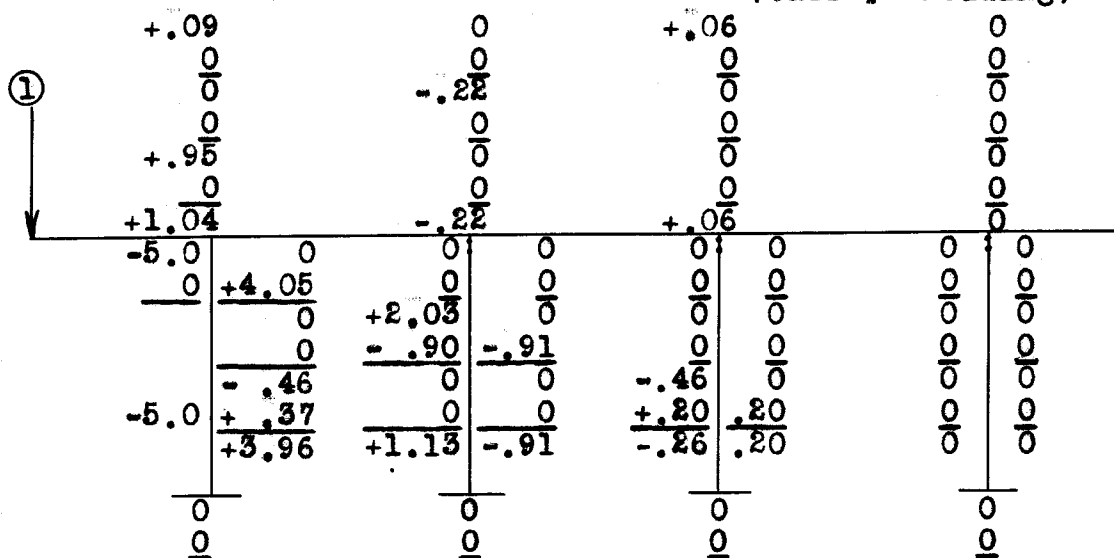
Assume column to be 2' by 2', $EI/L = 1/12(2)(2)^3/20$
 $= .065$ and $k = 1$

$$M = 1/8(w)(l) = 1/8(1)(10) = 1.25$$

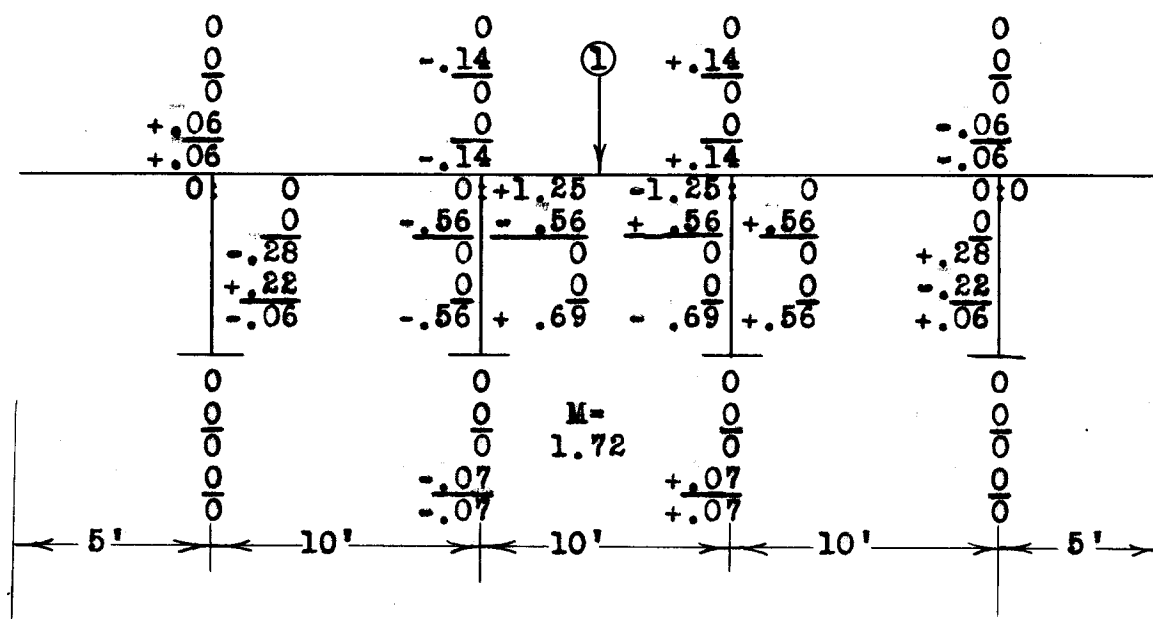
(Case #1 Loading)



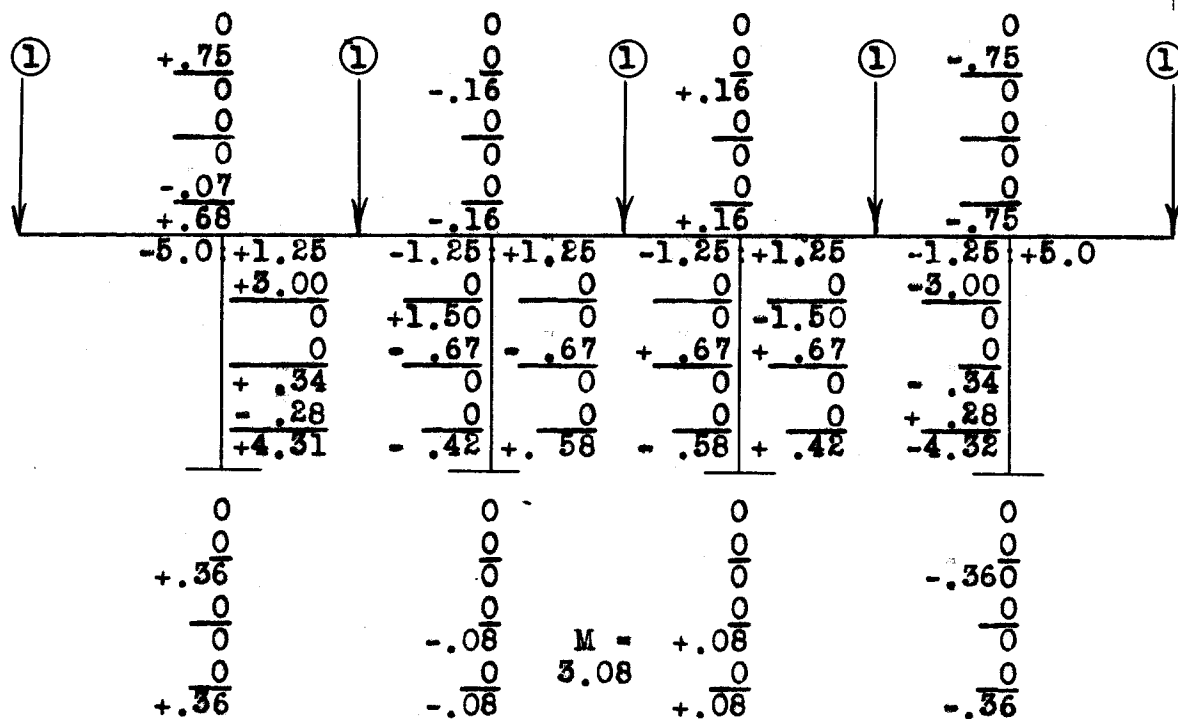
(Case #2 Loading)



(Case #3 Loading)



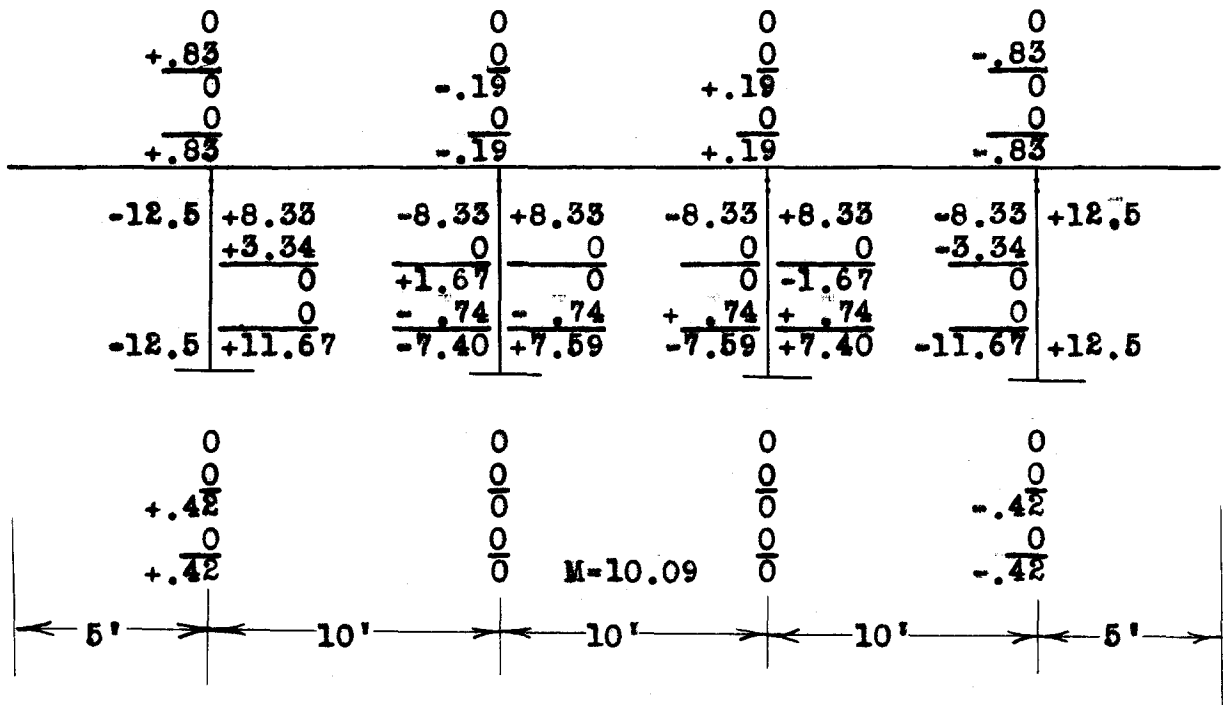
(Concentrated Dead Load)



(Uniform Dead Load)

1# per ft.

$$M = 1/12 w l^2 = 1/12 (1)(120) = 8.33$$



WORKING STRESSES

$$f_c = (.04)(2000) = 800$$

$$f_s = 18,000$$

$$n = 15$$

$$u = (.05)(2000) = 100$$

$$v(\text{no web reinforcement}) = (.03)(2000) = 60$$

$$v(\text{with web reinforcement}) = (.12)(2000) = 240$$

$$K(\text{over supports}) = 165$$

$$K(c.l.) = 138.7$$

FLOOR DESIGN

$$\begin{aligned} \text{Effective width for moment} &= (4/3)x, x = \text{span}/2 = 5/2 = 2.5 \\ &= (4/3)2.5 = 3.33' \end{aligned}$$

$$\begin{aligned} \text{Effective width for shear} &= (4/3)x, x = 2.5d, \text{ assume } d = 7" \\ &= (4/3)(2.5)(7) = 23.1" = 1.82' \end{aligned}$$

Loads

$$(\text{Moment}) \text{ Concentrated } P = (.4)(2000)(12) = 9600\#$$

$$Q_1 = \text{Load per ft. bridge floor} = 9600/3.33 = 2890\#$$

$$W = \text{Uniform load}$$

$$\text{Wt. of slab} = (9/12)(150) = 112\#$$

$$\text{Wt. of pavement} = 20\#$$

$$W = 112 + 20 = 132\#$$

$$(\text{Shear}) \text{ Concentrated } P = 9600\#$$

$$Q_2 = \text{Load per ft. bridge floor} = 9600/1.82 = 5220\#$$

$$W = \text{Same as for moment} = 132\#$$

(Moments)

$$\text{Max. plus moment as shown} = +.74Q_1$$

$$\text{Max. neg. moment as shown} = -.703Q_1$$

$$\text{Max. plus moment} = .74(2890) + 1/24(5)^2(132) = 2275$$

$$\text{Adding impact} = 2275 + .38(2275) = +3140' / \#$$

$$\text{Max. negative moment} = .71(5220) + 1/12(5)^2(132) = -4075$$

$$\text{Adding impact} = 4075 + .38(4075) = -6620' / \#$$

(Shears) Max. shear when large wheel is at center of span.

$$V_o = (1/2)(Q_2) = 1/2(5220) = 2610 + 500 = 3100' / \#$$

$$\begin{aligned} V_s(\text{Max}) &= Q_2 + 1/2(W)(L) = 5220 + 1/2(132)(5) \\ &= 5500 + 700 = 6200' / \# \end{aligned}$$

(Depth of Floor Slab)

Moment

$$bd^2 = M/K \quad b = 12"$$

$$\begin{aligned} d &= \sqrt{M/K+b} \\ &= \sqrt{(6620)(12)/(139)(12)} \quad (\text{at c.l.}) \end{aligned}$$

$$d = 7" (\text{about})$$

$$d = \sqrt{(6620)(12)/(165)(12)} \quad (\text{at support})$$

$$d = 6+"$$

Shear

$$bd = V/v_j$$

$$\begin{aligned} d &= V/v_j b \\ &= 3100/(60)(7/8)(12) \quad (\text{at c.l.}) \end{aligned}$$

$$d = 5+"$$

$$d = 6200/630 \quad (\text{at support})$$

$$d = 9"$$

Use $d = 9"$, $d_1 = 2"$, $d+d_1 = 11"$ (slightly in excess of assumed value)

(Area of Steel)

$$A_s = M/f_s j d \quad (\text{at c.l.})$$

$$A_s = 3140/(18000)(.875)(7/12)$$

$$A_s = .28 \text{ square inches}$$

$$A_s = 6620/(18000)(.875)(7/12) \quad (\text{at support})$$

$$A_s = .14 \text{ square inches}$$

7/12 of steel at c.l. must be carried in bottom of slab at support. $(7/12)(.28) = .16$ square inches in bottom of slab over support.

(Bond)

$$\Sigma O = V/vjd$$

$$= 3100/(100)(.875)(7) \quad (\text{at c.l.})$$

$$\Sigma O = 5.06" \text{ in bottom}$$

$$\Sigma O = 6620/61.3 \quad (\text{at support})$$

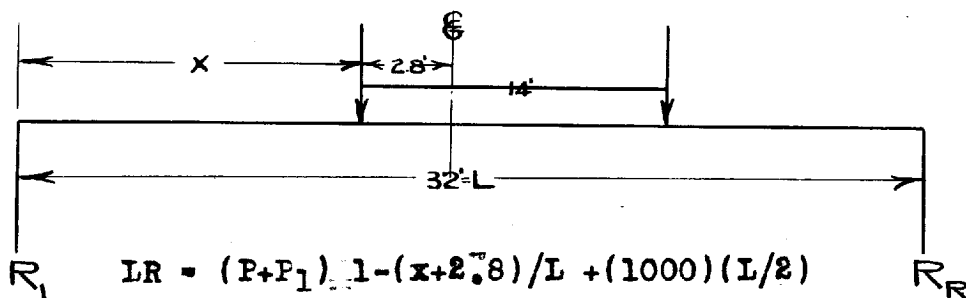
$$\Sigma O = 10.8" \text{ at top}$$

(Steel) Use $1/2" \phi - 12" \text{ c to c}$ in bottom from rail to rail.

Bend $5/8" \phi - 12" \text{ c to c}$ through inflection points so that the steel is at top over supports and in bottom at c.l. Add 2 - $5/8" \phi - 6" \text{ c to c}$ over supports and extend to point of inflection on each side. Inflection Point is $1/4(5) = 1.25'$ from c.l. of support.

FLOOR BEAM DESIGN

Each beam will carry 1 row of truck wheels plus 5' of floor slab



$$LR = (P+P_1) [1-(x+2.8)/L] + (1000)(L/2)$$

$$M = RX - 500X^2$$

$$\begin{aligned} \text{Max. shear at center} &= (P/2) + (2/32)P_1 \\ &= (9600/2) + (2/32)(2400) \\ &= 4950\# \end{aligned}$$

Reactions and moments are shown on next page.

X in Ft.	M in Ft. lbs.	R in lbs.
0	0	27,050
4	104,200	25,480
8	159,000	23,900
12	198,000	22,480
14 (max.)	216,800	21,640
14.5	207,000 _m	21,500
15	206,500	21,270
16	206,000	20,900

(Interior Beam Design)

"T" at c.l.

$$M/f_s b t^2 = 216,800(12)/(18000(60)(108)) = .022$$

$$f_c/f_s = 800/18000 = .045$$

Beam will be designed as rectangular beam.

$$b d^2 = M/K$$

$$60 d^2 = 216,800(12)/139$$

$$60 d^2 = 18,700$$

$$d^2 = 312$$

$$d = 18"$$

(Rectangular Beam at Support)

$$b d = V/v_j$$

$$b d = 27050/96(.875)$$

$$b d = 282"$$

Use $b = 14"$ (width of T Beam stem at c.l.)

$$d = 20"$$

(Steel at c.l.)

$$A_s = M/f_s j d =$$

$$= 216,800(12)/18,000(.875)(18)$$

$$A_s = 9.2 \text{ square inches}$$

Use 6 - 1 1/4" square bars in 2 rows

$$d+d' = 20"$$

$$18+2 = 20"$$

(Stirrups)

Use 3/8" ϕ U Stirrups

$$V_o = vbjd$$

$$= (60)(14)(.875)(21)$$

$$V_o = 15,500$$

$$V_s(\text{at support}) = 27,050 - 15,500 = 11,550 \text{ and } s = 5 1/2"$$

$$\text{Max. allowed spacing} = (45/90+10)(d-18) = 8"$$

$$\text{Spacing changes to } 6" \text{ when } V_s = 10,000$$

This occurs 1.5' from c.l. of support

$$\text{Spacing changes to } 7" \text{ when } V_s = 7,750$$

This occurs 24" from c.l. of support

$$\text{Spacing changes to } 8" \text{ when } V_s = 6,500$$

This occurs 3.6' from c.l. of support

These spacings will be used as shown in diagram except where they can be omitted on account of bent up bars.

Stirrups to 7.7' of c.l.

(Weight of Beam)

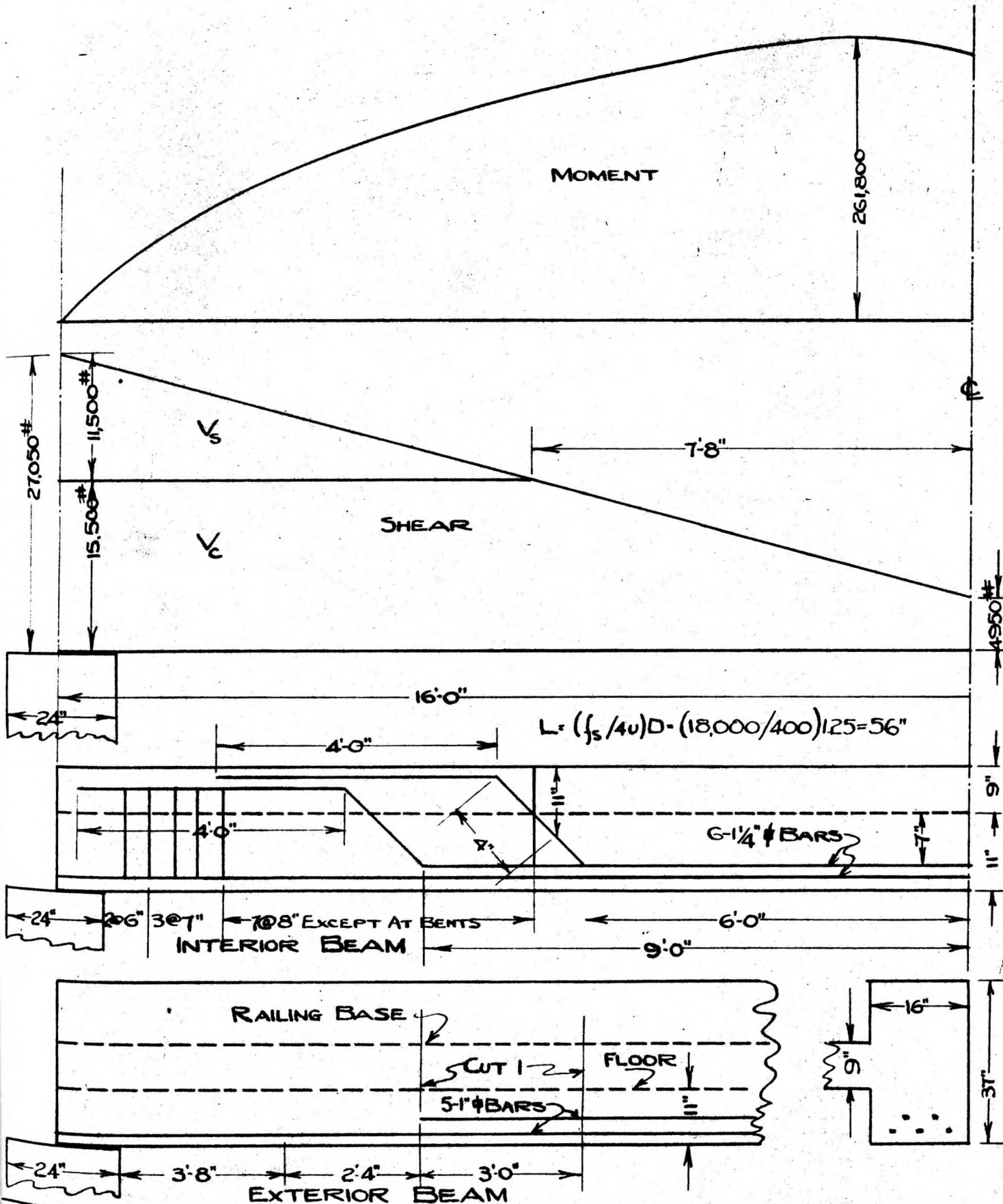
$$[(14/12)(20-9)/12]150 = 161\# \text{ per ft. (assumed 150\#)}$$

(Exterior Beam Design)

This beam will carry 3' of floor load having a weight of $(9/12)(3)(150) = 350\#$. Assume railing weighs 300#, giving $W = 650\#$. This load nears that as used for interior beam design (slightly less) and the same shears and moments as used for interior beam design will be used. Beam shall be designed as rectangular, the railing base being a part of the beam.

$$bd^2 = 18,700, b = 16 \text{ and } d = 34, d' = 3"$$

MOMENT, SHEAR & BENDING DIAGRAMS FOR BEAMS



$$V_c = v_d j b = (60)(16)(.875)(34) = 28,600\#$$

$$V_s = 27,050 - 28,600 = 1,550\# \text{ (no stirrups required)}$$

(Area of Steel)

$$A_s = 207,000(12)/(18,000)(.875)(34) = 4.63 \text{ sq. ins.}$$

Use 5-1" square bars

GIRDER DESIGN

Four columns spaced 10' on centers, with a cantilever overhang of 5' on each side. The girder will span these columns.

(Uniform Load)

(W) Girder Weight = 800# per ft. (assumed)

(B) Concentrated Dead Load, Due to weight of floor, pavement and stems of floor beams. The girder is assumed to support the weight of 16' or one half the space of floor in each direction. Total 32'

Wt. of bridge floor = $(32)(8)(.75)(150) = 28,800\#$

Wt. of pavement = $(32)(8)(.75)(25) = 4,800$

Wt. of stem = $(14/12)(11/12)(32)(150) = 5,130$

Total (B) = 38,730#

(A) Concentrated Live Load

$$P + (P_1)(L-a/L) + (P_1)(L-a/L) = 9,600 + 2,400(.97) = 11,900\#$$

Maximum moment coefficients as given by the moment charts are tabulated below:

Load	Over 1st Col.	C.L. Span	Over 2nd Col.
800 (W)	-12.5	10.09	-7.4
38,730 (B)	- 5.0	3.08	-0.58
11,900 (A)	<u>- 5.0</u>	<u>1.72</u>	<u>-0.42</u>
Moments =	-263,150	152,216	-49,650

Using b as 24" and substituting in the formula $bd^2 = M/K$,
 $d = 28"$ over the 1st column, 23" at c.l. of span and 13"
 over the 2nd column.

(Shears)

$$v = 60 \quad d = V/(60)(.875)(24) = V/1260$$

$$\text{At the end of overhang} \quad V = 11,900 + 38,730 = 50,630, \quad d = 40"$$

$$\text{Left of 1st Column} \quad V = 50,630 + 4,000 = 54,630, \quad d = 43"$$

$$\text{Right of 1st Column} \quad V = .5(50,630) + 8,000 = 33,315, \quad d = 26"$$

$$\text{Left of 2nd Column} \quad V = .5(50,630) + 8,000 = 33,315, \quad d = 26"$$

$$\text{C.L. of Span} \quad V = .5(50,630) = 25,315, \quad d = 20"$$

Stirrups will be used in the cantilever overhangs, d being used
 as 28" and $d' = 3"$

Check for shear at left of support in overhang. Beam (with
 stirrups) will carry $(120)(24)(.875)(28) = 70,050\#$. Shear is
 50,630, so 120 is not exceeded.

Check for girder weight

$$(24/12)(31/12)(150) = 780\# \quad (800\# \text{ was assumed})$$

(Steel Required)

$$A_s = M/f_s j d = .00064M/d \quad O = V/u_j d$$

$$d = 28" \text{ Over 1st Col.} \quad A_s = 6 \text{ sq. in.} \quad O = 22.4" \quad V = 54,630$$

$$d = 23" \text{ At c.l. Span} \quad A_s = 4.23 \quad O = 12.5" \quad V = 25,315$$

$$d = 13" \text{ Over 2nd Col.} \quad A_s = 2.45 \quad O = 14.7" \quad V = 33,315$$

$$\text{Over 1st Col. use 6-1" sq. bars, } A_s = 6 \text{ sq. ins., } O = 24"$$

$$\text{In C.L. Span use 5-1" sq. bars, } A_s = 5 \text{ sq. in., } O = 20"$$

$$\text{Over 2nd Support use 4-1" sq. bars, } A_s = 4 \text{ sq. in., } O = 16"$$

All bars over 1st support will be carried along the top of
 the girder over the 2nd support except 2-1" square bars.

These bars will be cut off at a distance $L = (f_s/4u)D$,

$L = (18,000/400)1 = 45"$ from right edge of first column.
 These 4 bars will run the entire distance of the girder.
 1-1"sq. bar is added at c.l. and is carried a distance of
 4'0" on each side of c.l. Two 1"sq. bars will be carried
 completely around the girder on the outer edges.

(Stirrups in Cantilever Overhang)

Use 3/8" ϕ U stirrups

At left of 1st column $V_c = (60)(24)(.875)(28) = 35,300$

$V_s = 54,630 - 35,300 = 19,330$ and $s = 5"$ (from diagram)

At end of overhang $V_c = 50,630$

$V_s = 50,630 - 35,300 = 15,330$ and $s = 6"$

Put 1st stirrup 6" from end, space 4 at 6" and 4 at 6",
 none being used over the support.

COLUMN DESIGN

Columns for this bridge will be designed as eccentric

(Interior Column Loadings)

Case #1 (see accompanying diagrams for case loadings)

Load (P) at end of girder, over 1st column, and
 c.l. columns.

Load = $2.5(A) = 2.5(11,900) = 29,700$

$2.5(B) = 2.5(38,730) = 96,700$

$10(W) = 10(780) = 7,800$
 Total $\underline{134,400\#}$

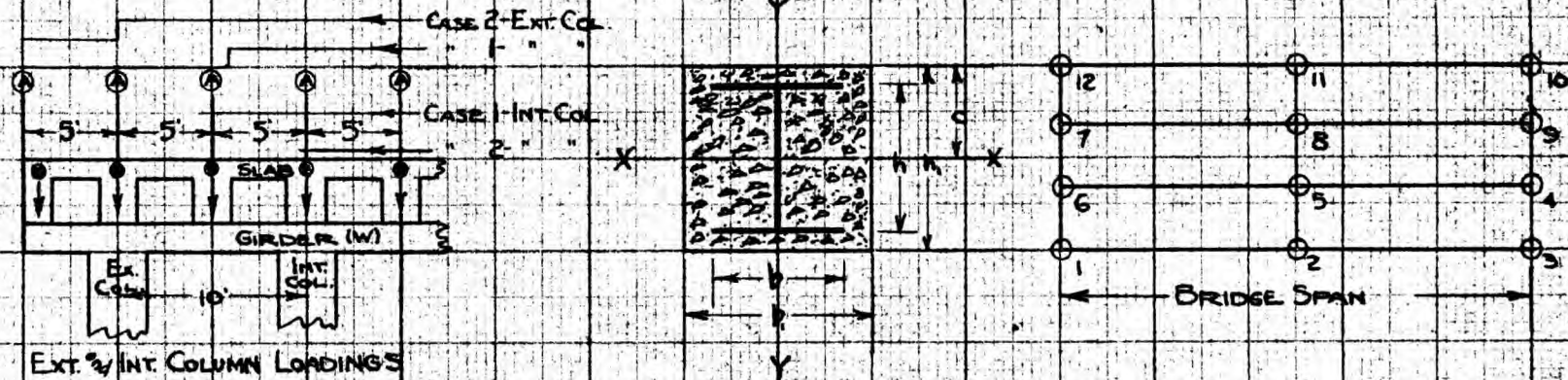
Eccentricity Moment = $2.5(A+B) - 5(A+B)$

= $2.5(50,630) - 5(50,630)$

= $-126,000' / \#$

$X_o = 126,000 / 134,000 = .933 \text{ ft.} = 11.2"$

COLUMN DESIGN



COLUMN NUMBER	LOADING CASE	N	X ₀	M	SECTION	A _s	b	b	d	d ₁	I _{x,x}	I _{y,y}	EQUIVALENT AREA	I	f _c	f _s
2, 11	2	212,600	2.11	4,480,000	BETH-H-1007	209	10.12	14.5	10.38	14.5	405	136.5	482	5450	784	6350
5, 8	2	171,775	4.12	705,000	" H-8035	1030	8.0	12.0	8.0	12.0	123	41.1	276	2308	700	1400
1, 2, 10, 3	2	122,800	23.8	3,150,000	" H-8044	1296	8.00	13.0	8.25	13.0	159	53.4	338	3180	705	7000
6, 4, 7, 9	2	98,350	5.13	3,040,000	" H-80235	685	6.5	12.0	7.75	12.0	16.1	16.8	233	1980	512	1350

N FOR CASE 1 (EXT. COL.) = $25 \cdot A + 25 \cdot B + 10 \cdot W$ — $M = 5.0(A+B) + 5(A+B)S$ — $f_c = 2000$
 $" " " 2 (" " ") = 2 \cdot A + 25 \cdot B + 10 \cdot W$ — $M = 5.0(A+B) + 5(B)S$ — $n = 15$
 $" " " 1 (INT. COL.) = 2 \cdot A + 2 \cdot B + 10 \cdot W$ — $M = \text{NONE}$ — $f_c = 800(\text{all})$
 $" " " 2 (" " ") = 15 \cdot A + 2 \cdot B + 10 \cdot W$ — $M = 5.5(A+B)S + 5(B)S$ — $f_s = 15,000(\text{all})$
 EQUIVALENT AREA = $(A_{\text{steel}} - A_s) + (n-1)A_s$ — $k = .429$
 $I = I_c + nI_s = \frac{1}{2}bd^3 + nI_s$ — $X_0 = M/N$
 f_c (load inside middle third) = $N/A + M_c/I$
 f_s (" outside ") = $N/A + \frac{M(EI)}{nI}$
 $f_s = N/A - \frac{M(1-k)d}{I}$

NOTE - THE LOADING USED IS THAT WHICH PRODUCES THE GREATEST ECCENTRICITY

Case #2 Load at end of girder and over 1st column

$$\text{Load} = 2(19,900)(A) = 39,800$$

$$2.5(33,600)(B) = 84,000$$

$$\begin{array}{rcl} 10(780)(W) & = & 7,800 \\ \text{Total} & & 131,600\# \end{array}$$

$$\begin{aligned} \text{Eccentricity Moment} &= 5(A+B+5W) - 2.5(B) \\ &= 154,950' / \# \end{aligned}$$

$$\begin{aligned} X_o &= 154,950 / 131,600 \\ &= 1.17 \text{ ft.} = 14.1" \end{aligned}$$

(Design) Working stresses - $f_o = 600$, $f_s = 18,000$, $n = 15$, $d'/t = .10$

Case #2 loadings control

$$N = 131,600, X_o = 14.1", p = 0.04, t = 25"$$

$t/X_o = 25/14.1 = 1.77$, from which $NX_o/f_o b t^2 = .24$ (from diagram), from which $b = 21"$ (b will be used 24", same as for girder)

$$A_s = p b t = .04(24)(25) = 24 \text{ sq. in. Use } 8-1\frac{1}{4}" \text{ sq.}$$

bars in each face, spaced at 3" c to c. This gives an area of $16(1.563) = 25 \text{ sq. in.}$ Use $1\frac{1}{4}" \phi$ bars at 8" c to c as ties.

(Interior Column Loadings)

Case #1 Load over the column, between the columns on each side

$$\text{Load} = 2(A) = 2(19,900) = 39,800\#$$

$$2(B) = 2(33,600) = 67,200$$

$$\begin{array}{rcl} , 10(W) & = & 2(3,900) = 7,800 \\ \text{Total} & & 114,800\# \end{array}$$

Eccentricity moment = none

Case #2 Load over the column and between the column on one side.

$$\begin{aligned}
 \text{Load} &= (1.5)(A) = (1.5)(19,900) = 29,100 \\
 (2)(B) &= (2)(33,600) = 67,200 \\
 (10)(W) &= (10)(780) = 7,800 \\
 \text{Total} &= 104,100\#
 \end{aligned}$$

$$\begin{aligned}
 \text{Eccentricity Moment} &= -2.5(A+B) + 2.5(B) \\
 &= 49,000' / \#
 \end{aligned}$$

$$X_o = 49,000 / 104,100 = .47' = 5.65''$$

(Design) Case #2 Loadings Control

$$\begin{aligned}
 N &= 104,100, X_o = 5.65'', p = 0.04, t = 14'', n_p = .60 \\
 t/X_o &= 14/5.65'' = 2.48 \text{ in.}, d_1/t = 0.10 \text{ (from diagram)}, \\
 NX_o/f_c b t^2 &= .23 \\
 104,100(5.65)/600(b)(14)^2 &= .23 \\
 b &= 22'' \text{ (24'' will be used)}
 \end{aligned}$$

Check for Case #1 Loadings

$$\begin{aligned}
 N &= 108,560, X_o = 0, p = .04 \\
 f_c &= N/A(1+p(n-1)) \\
 &= 108,560 / (14)(24)(1+.28) \\
 &= 252\# \text{ (Allowable is } 400\# \text{ so design need not be changed)}
 \end{aligned}$$

(Steel) $A_s = p b t = (.04)(14)(24) = 13.4 \text{ sq. in.}$

Use 5-1 $\frac{3}{4}$ '' sq. bars in each face (b)

This gives an area of 10(1.56) = 15.6 sq. in.

Use 1/4'' ϕ ties 8'' c to c.

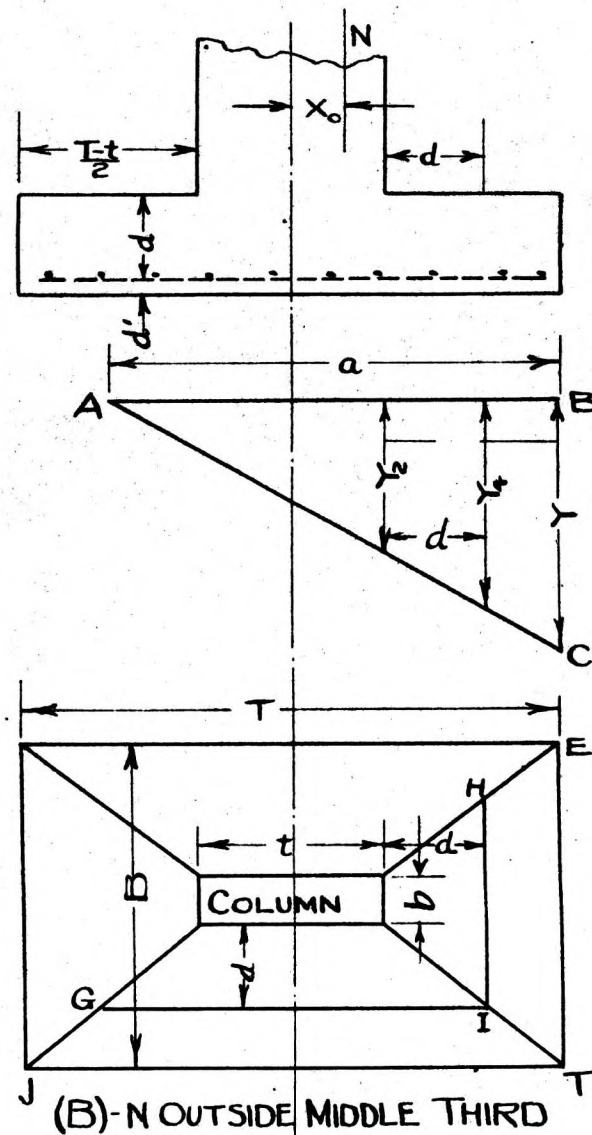
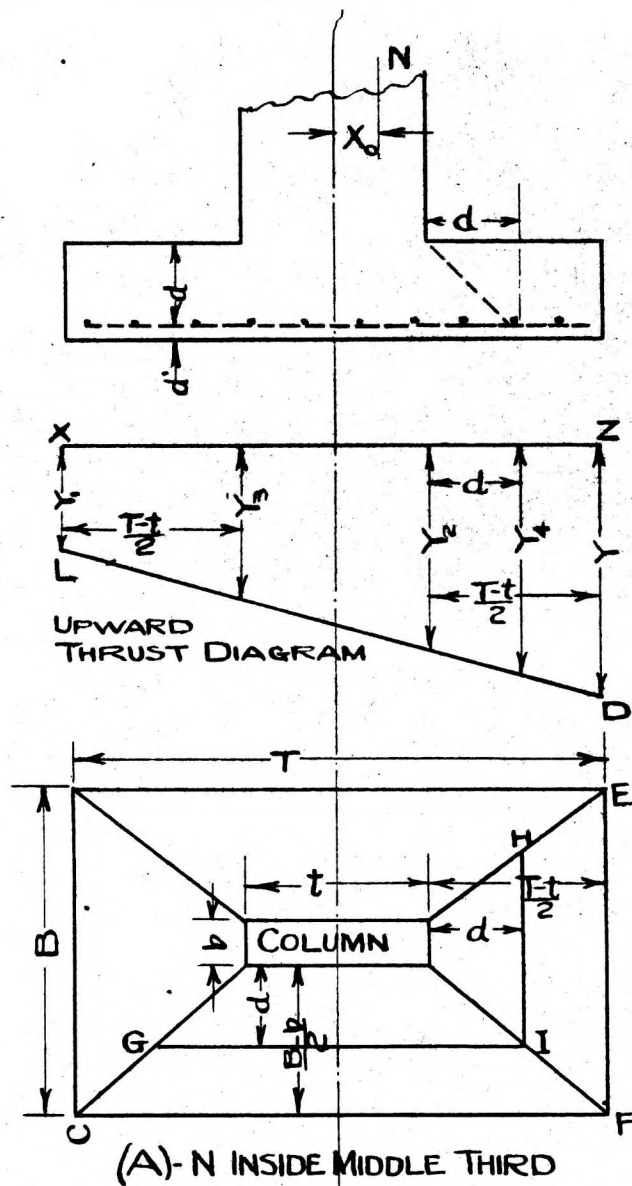
COLUMN FOOTINGS

Columns 20' Long

(Interior Column Footing)

(See accompanying column footing diagrams for notations)

$$\begin{aligned}
 t &= 1.17', b = 2.0', M(\text{column}) = 49,000 \text{ ft. lbs.}, N = 104,100\# \\
 N &= 104,100 + 20(2.0)(1.17)(150) = 111,100\#, \text{ Assumed wt. of footing (W) } = 5,000\#, \\
 X_o &= 49,000 / 111,100 = .44' \text{ (Assumption is made)}
 \end{aligned}$$



ECCENTRIC COLUMN FOOTINGS

that the load is inside the middle third), that is, T is greater than $6(.44) = 2.64'$

$$BT = .0005(N/L+W/L)(1+6X_o/T) = .0005(116,100/5)(1+2.64/T)$$

$$= 11.6+306/T, \text{ or } BT^2 - 11.6T = 30.6. \text{ Make } B = 3+2 = 5'$$

$$\text{and } T = 1.17+3 = 4.17'$$

$$y = N/BT+6NX_o/BT^2 = 111,000/5(4.17)+6(111,00)(.44)/5(4.17)^2$$

$$= 9,930\# \text{ per sq. ft. (max. upward thrust)}$$

$$y_1 = N/BT-6NX_o/BT^2 = 2,270\# \text{ per sq. ft. (Min. upward thrust)}$$

$$y_2 = y-(y/2-y_1/2)(1-t/T) = 9,930-(7,660).5(1-1.17/4.17)$$

$$= 7,100\# \text{ per sq. ft.}$$

$$y_3 = y-(y/2-y_1/2)(1+t/T) = 9,930-4,700 = 5,170\# \text{ per sq. ft.}$$

$$P_1 = (\text{Punching shear along } b)$$

$$= (T-t/12) B(y_2+2y)+b(y+2y_2)$$

$$= (4.17/12-1.17/12) 5(7,100+2(9,930))+2(9,930+2(7,100))$$

$$= 45,765\#$$

$$M_1 = 3/8(P_1)(T-t) = 3/8(45,765)(4.17-1.17) = 51,500 \text{ ft. lbs.}$$

$$P_2 = (B/12-b/12) T/2(y_1+y_2+y_3)+t/2(y_1+y_2+y_3)$$

$$= (5/12-2/12) .5(4.17)(9,930+7,100+4,700)+.5(1.17)(21,730)$$

$$P_2 = 14,450\#$$

$$M_2 = 3/8(P_2)(B-b) = 3/8(14,450)(3) = 16,250 \text{ ft. lbs.}$$

$$d_{1(B)} = \sqrt{M_1/Kb} = \sqrt{51,500/139(2)} = 13" \quad (v = .06f_c = .06(2,000) = 120)$$

$$d_{1(B)} = P_1/12bv = 45,765/12(2)(120) = 16"$$

$$d_{1(T)} = \sqrt{M_2/Kt} = \sqrt{16,250/139(1.17)} = 10"$$

$$d_{1(T)} = P_2/12tv = 14,450/12(1.17)(120) = 8"$$

$$\text{Use } d = 16" = 1.33' \quad GI = 3.87', HI = 4.66' (\text{by graphics})$$

$$y_4 = y-(y-y_1)(T/2-t/2)-d T = 9,930-(9,930-2,270)(.18) = 8,950\#/sq.f$$

$$V_1(\text{along HI}) = \text{Area ABHI}(y/2+y_4/2) = .$$

$$V_1 = .5(4.66+5.0)(.17)(9,930+8,950)0.5 = 7,830\#$$

$$\begin{aligned} V_2(\text{along GI}) &= \text{Area ACGI}(y/2+y_1/2) = \\ &= .5(3.87+4.17).17(9,930+2,270)0.5 = 4,150\# \end{aligned}$$

$$v = V_1/12(HI)(jd) = 7,830/12(4.66)(.875)(16) = 10\#/sq. ft.$$

$$v = V_2/12(GI)(jd) = 4,150/12(3.87)(.875)(16) = 7\#/sq. ft.$$

$$\text{Allowable } v \text{ is } .02(2,000) = 40\#/sq. ft.$$

(Steel)

$$\begin{aligned} A_s(\text{T direction}) &= M/f_s jd = 51,500(12)/18,000(.875)(16) \\ &= 2.45 \text{ sq. in.} \end{aligned}$$

$$\begin{aligned} b+2d &= 2+2.66 = 4.66. \text{ Steel is distributed over } .5(5+2)+1.33 \\ &= 4.88' \end{aligned}$$

$$A_s \text{ per ft.} = 2.45/4.88 = .50 \text{ sq. in.}$$

$$\Sigma O = 7,830/75(.875)(16) = 6.5"$$

$$\Sigma O \text{ per ft.} = 6.5/4.88 = 1.33". \text{ Use } 7/8" \phi \text{ bars at } 12" \text{ c to c.}$$

$$A_s(\text{B direction}) = M_2/f_s jd = 16,250(12)/18,000(.87)(16) = .77 \text{ sq. in.}$$

$$\begin{aligned} t+2d &= 1.17+2.66 = 3.83'. \text{ T is greater than } t+2d, \text{ Steel is dis-} \\ &\text{tributed over } .5(4.17+1.17)+1.33 = 4.0' \end{aligned}$$

$$A_s \text{ per ft.} = .78/4 = .195 \text{ sq. in.}$$

$$\Sigma O = 4,150/75(.875)(16) = 3.95"$$

$$\Sigma O \text{ per ft.} = 3.95/4 = 1". \text{ Use } 1/2" \phi \text{ at } 12" \text{ c to c}$$

(Exterior Column Loadings) See accompanying diagrams for notation

$$t = 25" = 2.08', \quad b = 24" = 2.0', \quad N = 131,600\#,$$

$$M(\text{column}) = 154,950 \text{ ft. lbs., } N(\text{for footing}) =$$

$$131,600+(20)(2.08)(2.0)(150) = 143,600\#. \text{ Assume } W = 5,000\#$$

$$X_0 = 154,950/143,600 = 1.08'. \text{ Assume } N \text{ to be outside middle third.}$$

$$L = 5 \text{ tons per sq. ft.}$$

$$B(T/2-X_0) = N/3,000L+W/3,000L,$$

$$B(T/2-1.08) = 131,600+5,000/3,000(5), \quad B(T/2-1.08) = 9.06$$

$$\text{Make } B = 4+2 = 6' \text{ and } T = 4+2.08 = 6.08'$$

$$y = 2N/aB, \quad a = (.001)N/LB = (.001)131,600/5(6) = 4.37'$$

$$= (2)131,600/(4.37)6 = 10,100\# \text{ per sq. ft.}$$

$$y_2 = y(1 - (T-t)/2a)$$

$$= 10,100(1 - (6.08 - 2.08)/(8.74)) = 5560\# \text{ per sq. ft.}$$

$$P_1 = (T-t)12 B(y_2 + 2y) + b(y + 2y_2)$$

$$= .336(5560 + 2(10,100) + 2(10,100) + 2(5560)) = 65,000\#$$

$$M_1 = 3/8(P_1)(T-t) = 3/8(65,000)(4) = 112,500 \text{ ft. lbs.}$$

$$P_2 = (T/6 + t/6)(B-b)y$$

$$= (6.08/6 + 2.08/6)(6-2)10,100 = 55,000\#$$

$$M_2 = 3/8(P_2)(B-b) = 3/8(55,000)(4) = 82,500 \text{ ft. lbs.}$$

$$d_1 = \sqrt{M_1/Kb} = \sqrt{112,500/139(2)} = 20"$$

$$d_1 = P_1/12v_p b = 65,000/12(120)2 = 22"$$

Use $d = 22"$

$$d_2 = \sqrt{M_2/Kt} = \sqrt{82,500/139(2.08)} = 17"$$

$$d_2 = P_2/12v_p t = 55,000/12(120)2.08 = 19"$$

$$y_4 = y \left[1 - (.5(T-t)-d)/a \right] = 10,100 \left[1 - (.5(4)-1.8)/4.37 \right] = 9700\#/sq$$

$$HI = 5.87" \text{ and } GI = 5.9 \text{ (by graphics)}$$

$$V_1 = (5.87+6) \cdot 5(.21)(10,100+9700) \cdot 5 = 12,375\#$$

$$V_2 = (5.9+6) \cdot 5(.21)(10,100) \cdot 5 = 6,300\#$$

$$v_1 = V/bjd = 12,375/5.87(.875)22(12) = 9\# \text{ sq. in.}$$

$$v_2 = V_2/bjd = 6300/5.9(.875)22(12) = 5\# \text{ sq. in.}$$

$$\text{Allowable } v = .02(2,000) = 40\# \text{ sq. in.}$$

$$A_s(T \text{ direction}) = M_1/f_s jd = 112,500(12)/18,000(22) \cdot 875 = 4 \text{ sq. in.}$$

$$b+2d = 2+2(1.83) = 5.66 \text{ so } B \text{ is greater than } b+2d \text{ and steel is distributed over } .5(6+2)+2 = 6'$$

$$A_s \text{ per ft.} = 4/6 = .65 \text{ sq. in.}$$

$$\leq 0 = V_1/ujd = 12,375/75(22) \cdot 875 = 8.6"$$

$$\leq 0 \text{ per ft.} = 8.6/6 = 1.43. \text{ Use } 1" \phi \text{ bars at } 12" \text{ o to o}$$

$$A_s(\text{B direction}) = M_2 / f_s j d = 82,500(12) / 18,000(.875) 22 = 2.77 \text{ sq. in.}$$

$$t + 2d = 2.08 + 2(1.83) = 5.74. \text{ Steel is distributed over}$$

$$.5(6.08 + 2.08) + 2 = 6.08'$$

$$A_s \text{ per ft.} = 2.77 / 6.08 = .45 \text{ sq. in.}$$

$$\Sigma O = V_2 / u j d = 6,300 / 75(22) .875 = 4.73$$

$$\Sigma O \text{ per ft.} = 4.73 / 6.08 = .78" \text{ Use } 1" \phi \text{ bars at } 12" \text{ c to c.}$$

COST OF REINFORCED CONCRETE BRIDGE

Quantities will be computed and the following prevailing prices will be used in making the estimate of the total cost:

Concrete in place	\$20.00 cu. yd.
Reinforcing steel in place	0.07 per pound
2" Wearing surface	0.60 per sq. yd.
Forms	0.25 per sq. ft. exposed
Excavation	1.00 cu. yd.

It is assumed throughout this Thesis that the only excavation necessary is that for the center footings.

(Floor slab)

Cu. yds. concrete = $.75(37.30)128/27 = 132.5$ cu. yds.

Re Steel = $60(128).667 = 5,130\#$

= $128(37.30).667 = 3,160\#$

= $128(41)1.043 = 5,560\#$

(7 Interior Beams)

Concrete = $7(1)1.2(128)/27 = 40$ cu. yd.

Re Steel = $42(135)5.312 = 30,200\#$

Stirrups = $616(4).375 = 924\#$

(2 Exterior Beams)

Concrete = $2(128)3(1.35)/27 = 38.4$ cu. yd.

Re Steel = $6(128)3.4 = 2,160\#$

= $2(72)3.4 = 490\#$

= $2(48)3.4 = 326\#$

(2 Hand Railings)

Concrete = $(7/12)3.2(128)2/27 = 17.7$ cu. yd.

Re Steel = $256(6).667 = 1,020\#$

= $10(128).667 = 885\#$

(5 Girders)

Concrete = $5(41.2)2(31/12)/27$ = 47.2 cu. yd.
 Re Steel = $(10)(85)(3.4)$ = 2,890#
 = $(30)(40)(3.4)$ = 4,080#
 = $(15)(8)(3.4)$ = 408#
 Stirrups = $(90)(5.5)(.375)$ = 185#

(10 Exterior Columns)

Concrete = $(10)(2)(2.08)(20)/27$ = 30.8 cu. yd.
 Re Steel = $(160)(23)(4.173)$ = 15,300#
 = $(30)(7)(.167)$ = 35#

(10 Interior Columns)

Concrete = $(10)(2)(1.2)(20)/27$ = 17.8 cu. yd.
 Re Steel = $(100)(22)(4.173)$ = 9,160#
 = $(30)(5)(.167)$ = 25#

(10 Exterior Column Footings)

Concrete = $(10)(6)(6)(1.85)/27$ = 24.6 cu. yd.
 Re Steel = $(240)(5)(2.67)$ = 3,200#

(10 Interior Column Footings)

Concrete = $(10)(1.3)(4.5)(4.1)/27$ = 8.88 cu. yd.
 Re Steel = $(200)(4)(2.044)$ = 1,630#

(Forms)

12,500 ft. of exposed area and 7 m. W. f. lumber necessary
 for additional bracing of forms.

(Excavation)

Interior Footings(6) = $(6)(4.1)(4.5)(1.3)/27$ = 5.3 cu. yd.
 Exterior Footings(6) = $(6)(6)(6)(1.85)/27$ = 15.2 cu. yd.

(Pavement)

$(128)(38)/9$ = 540 sq. yd.

(Totals and Cost)

Concrete	- 365.8 cu. yd. at \$20.00	- \$7,316.00
Re Steel	- 87,188# at \$0.07	- 6,103.00
Wearing surface	- 540 sq. yd. at \$0.60	- 324.00
Excavation	- 20.5 cu. yd. at \$1.00	- 20.50
Forms	- 12,500' exposed area at \$0.25	- 3,125.00
	- 7 m. b. f. at \$13.00	- <u>91.00</u>
	Total	- \$16,979.50

ENCASED STEEL TYPE BRIDGE

Span - 2 at 64 ft. - 128 ft.

DESCRIPTION OF BRIDGE

General Description- This bridge shall be designed as an encased steel beam bridge made up of two sections at 64 ft., giving an overall length of 128 ft. as desired. Roadway shall be 40 ft. c to c of handrailing, the same as for the preceeding bridge. The same floor system will also be used here as the flange of a "T" beam. Concrete will be built up about the steel "I" section into a "T" section. Maximum moments and shears for each section will first be computed, a steel section chosen and built up, the allowable moment for this section determined and if equal to or greater than the former, will be used for that member. The same handrailing as used for the reinforced concrete bridge will also be used here. Encased girders and columns will also be used for this bridge. In the construction of such a bridge, the steel members would be bolted in place as a rigid frame structure, the forms hung from these members and the concrete poured.

General Dimensions- Same as for the reinforced concrete bridge except for the section length of 64 ft. in place of 32 ft.

Note- Moments on the slabs and girders of these first two bridges are identical except for the loadings. These moment coefficients are determined by the Hardy Cross method and these calculations are to found at the beginning of the first bridge.

DESIGN OF INTERIOR FLOOR BEAMS

Maximum moment for the floor beams will occur when the bridge is loaded as shown in the diagram on the following page, this maximum occurring under the wheel marked P. Referring to the diagram:

$$R_L = [(5.5 \cdot 2400) + (19.5 \cdot 9600) + (38.5 \cdot 2400) + (40.5 \cdot 9600)]/64 \\ = 10,600\#$$

$$M_P = [(10,600 \cdot 25.5) - (14 \cdot 9600)]/12 = 1,630,800 \text{ in. lbs.}$$

$$R_L (\text{due to dead load}) = (64)1000/2 = 32,000\# \\ (1000 = \text{wt. of pavement, stem, etc.})$$

$$M_P = [(32,000 \cdot 25.5) - (25,500 \cdot 22.75)]/12 = 2,850,000 \text{ in. lbs.}$$

$$M(\text{max.}) = 1,630,800 + 2,850,000 = 4,480,800 \text{ in. lbs.}$$

(Design) See accompanying print for interior beam notations
Concrete will be built around the steel section in the form of a "T" beam, the floor slab forming the flange of the beam.
A Bethlehem 28" I at 112# will be investigated.

$$h = 28.25" \quad \text{flange width} = 10.065" \quad A_s = 32.95 \text{ sq. in.}$$

$$I_s = S \cdot n = 306.4(28.25/2) = 4350 \quad a = 7.5"$$

$$f_c' = 2000\# \quad f_c = .4(2000) = 800 \quad f_s = 18,000\#$$

$$n = 15 \quad k = .429 \quad d = 36.25"$$

$$kd = 15.5 \quad b_1 = 14.065"$$

$$b = 2nA_s/(kd)^2 \cdot (h/2 + a - kd) \\ = 2 \cdot 15(32.95)/(15.5)^2 \cdot (28.25/2 + 6 - 15.5) = 18.9"$$

$$I = I_s + A_s(h/2 + a - kd)^2 + b(kd)^3/3n \\ = 4350 + 32.95(14.12 + 6 - 15.5)^2 + 18.9(15.5)^3/45 = 6618$$

$$M = nf_c I/kd = 15(800)6618/15.5 = 5,130,000 \text{ in. lbs.}$$

$$1/8wl^2 = 5,130,000/12, \text{ and since } L = 64, w = 840$$

$$\dot{V} = .5(840)(64) = 26,900\#$$

$$v = \frac{V}{nI} \cdot (kd - a/2) = 6(26,900)/15(6,618) \cdot (15.5 - 6/2) \\ = 20\# \text{ per sq. in.}$$

Since these moments and shears are less than the allowable this section will be used.

EXTERIOR FLOOR BEAMS

This beam will carry 3 ft. of floor. The weight of the beam, floor and steel will be assumed as 1000# per ft., the same as for the interior beams. Also assuming that the beam will carry one row of truck wheels, gives the loading, moment, and shears the same as for the interior beam. This beam will be designed as a rectangular beam, the railing base being considered part of the beam.

(Design) See accompanying print for exterior beam notations.

A Bethlehem 28" I at 112# will be investigated.

$$h = 28" \quad a = 20" \quad d = 48"$$

$$\text{Flange width} = 10.065" \quad A_s = 32.95 \text{ sq. in}$$

$$I_s = S(n) = 306.4(14.18) = 4350 \quad f_c' = 2000\#$$

$$f_c = .4(2000) = 800\# \quad f_s = 18,000\#$$

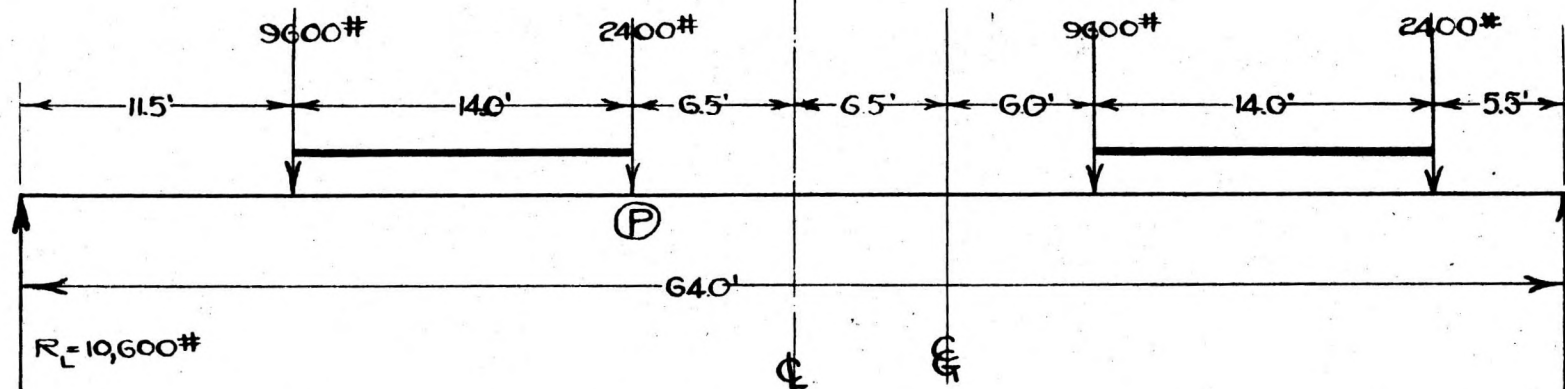
$$n = 15 \quad k = .429 \quad kd = 20.6$$

$$b = 2nA_s / (kd)^2 \cdot (h/2 + a - kd) \\ = 2(15)32.95 / (20.6)^2 \cdot (14 + 20 - 20.6) = 31.05$$

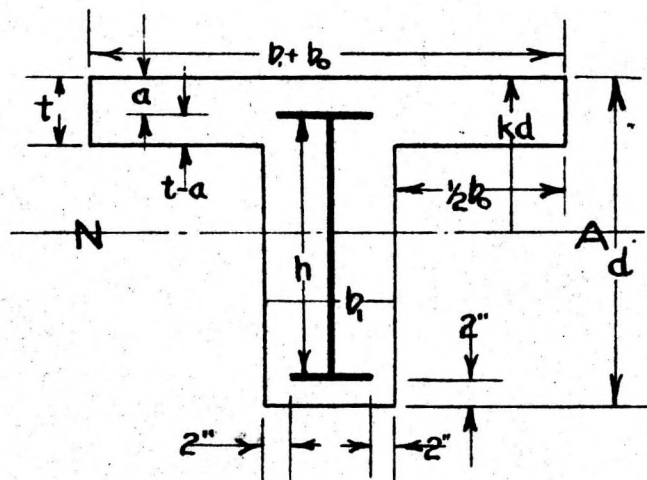
$$I = I_s + A_s(.5h + a - kd)^2 + b(kd)^3 / 3n \\ = 4350 + 32.95(14 + 20 - 20.6) + 31.05(20.6)^3 / 3(15) = \\ = 16,260$$

$$M = nf_c I / kd = 15(800)16,260 / 20.6 = 9,450,000 \text{ in. lbs.}$$

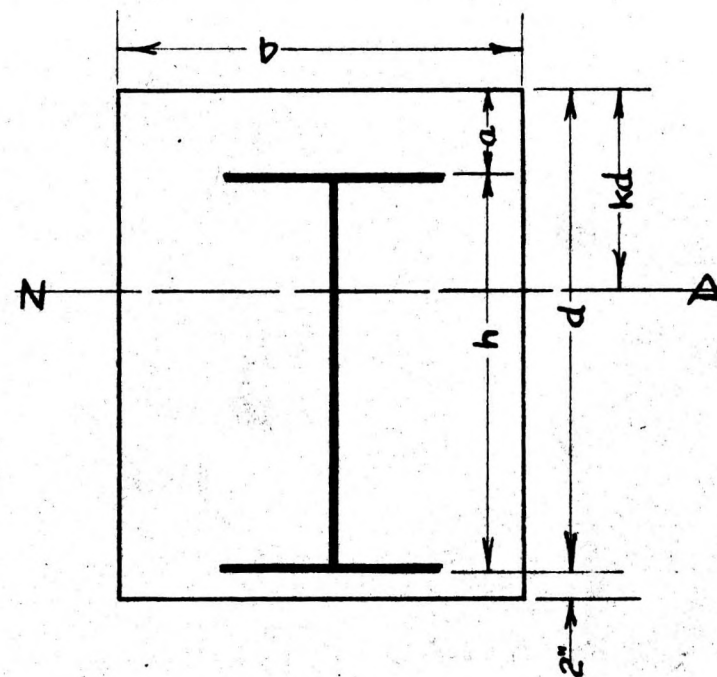
$$1/8wl^2 = 9,450,000/12, \text{ and since } l = 64, w = 1,540\#$$



(A)-BRIDGE LOADING FOR MAXIMUM MOMENT



(B) INTERIOR BEAM



(C) EXTERIOR BEAM AND GIRDER

$$V = 1/2wl = 1/2(1,540)64 = 49,200\#$$

$$v = aV/nI \cdot (kd - .5a)$$

$$= 20(49,200)/15(16,260) \cdot (20.6 - 10) = 38.8\# \text{ sq. in.}$$

This section will be used as the moment and shear is within the limits.

INTERIOR GIRDER DESIGN

Four columns will be spaced 10' apart. The girders will span these columns, with a cantilever overhang of 5' on each side.

(W) Uniform Load due to girder wt. = 1000#/' (assumed)

(B) Concentrated Dead Load

$$\text{Wt. bridge floor } (5)(64)(9/12)(150) = 36,000\#$$

$$\text{Wt. pavement } (5)(64)(2/12)(20) = 1,024\#$$

$$\text{Wt. stem } (14/12)(29/12)(62)(150) = 26,200\#$$

$$\text{Total} = 63,224\#$$

(A) Concentrated Live Load, due to truck wheels

(See accompanying print for loading)

$$= P + (P_1 \cdot 12)/64 + (P \cdot 31)/64 + (P_1 \cdot 45)/64$$

$$+ (P_1 \cdot 17)/64 + (P \cdot 31)/64 + (P_1 \cdot 50)/64$$

$$= P + P_1(12 + 45 + 17 + 50)/64 + P(31 + 31)/64$$

$$= 9600 + 2400(124)/64 + 9600(62)/64$$

$$= 23,550\#$$

The same coefficients as used for the preceding bridge girders will apply here, the only difference being a change in loadings.

The maximum coefficients occur over the first column and are as follows:

$$W = 12.5(1,000) = 12,500 \text{ ft. lbs.}$$

$$B = 5.0(63,224) = 316,120 \text{ " "}$$

$$A = 5.0(23,550) = 117,750 \text{ " "}$$

$$\text{Total} = 446,370 \text{ " "}$$

$$= 5,350,000 \text{ in. lbs.}$$

A Bethlehem 30" I at 121.0# will be investigated for this girder which must carry a moment of 5,350,000 in. lbs.

(Notations same as for exterior beam section)

$$\begin{aligned} h &= 30" & a &= 4" & d &= 34" \\ A_s &= 35.65 \text{ sq. in.} & f_s &= 18,000\# & f_c' &= 2000\# \\ f_c &= .4(2000) = 800\# & n &= 15 & k &= .429 \\ kd &= 14.6 & \text{Flange width} &= 10.50" \end{aligned}$$

$$I_s = S(n) = 351.3(15) = 5260$$

$$\begin{aligned} b &= 2nA_s / (kd)^2 \cdot (.5h + a - kd) \\ &= 2(15)35.65 / (14.6)^2 \cdot (15 + 4 - 14.6) = 22.1" \end{aligned}$$

$$\begin{aligned} I &= I_s + A_s(.5h + a - kd)^2 + b(kd)^3 / 3n \\ &= 5260 + 35.65(15 + 4 - 14.6)^2 + 22.1(14.6)^3 / 3(15) = 7,862 \end{aligned}$$

$$M = nf_c I / kd = 15(800)7,862 / 14.6 = 6,460,000 \text{ in. lbs.}$$

$$\begin{aligned} v &= aV / nI \cdot (kd - .5a) \\ &= 4(90,700) / 15(7,862) \cdot (14.6 - 2) = 38.8\# \text{ sq. in.} \end{aligned}$$

This section will be used as the moment and shear is within the allowable limits.

EXTERIOR GIRDER DESIGN

(W) Uniform load due to girder wt. = 900#/' (assumed)

(B) Concentrated Dead Load

$$\text{Wt. bridge floor } (5)(32)(9/12)(150) = 18,000\#$$

$$\text{Wt. pavement } (5)(32)(2/12)(20) = 1,024$$

$$\begin{aligned} \text{Wt. stem } (14/12)(29/12)(30)(150) &= 13,100 \\ \text{Total} &= 32,124\# \end{aligned}$$

(A) Concentrated Live Load, due to truck wheels

(See accompanying print for loading)

$$\begin{aligned} &= P + (P_1 \cdot 17) / 64 + (P \cdot 31) / 64 + (P_1 \cdot 50) / 64 \\ &= 9600 + (2400 \cdot 17) / 64 + (9600 \cdot 31) / 64 + (2400 \cdot 50) / 64 \\ &= 16,763\# \end{aligned}$$

Maximum moments:

$$W = -12.5(900) = 11,250$$

$$B = -5.0(32,100) = 160,500$$

$$A = -5.0(16,763) = 83,815$$

$$\begin{aligned} \text{Total} &= 255,565 \text{ ft. lbs.} \\ &= 3,060,000 \text{ in. lbs.} \end{aligned}$$

A Bethlehem 30" I at 110.0# will be investigated for this section. (Refer to previous print for notations)

$$h = 30"$$

$$a = 4"$$

$$d = 34"$$

$$I_s = S(n) = 314.8(15) = 4720$$

$$A_s = 32.45 \text{ sq. in.}$$

$$\text{Flange width} = 10.47"$$

$$f_o' = 2000\#$$

$$f_s = 18,000\#$$

$$f_o = .4(2000) = 800\#$$

$$n = 15$$

$$k = .429$$

$$kd = 14.6$$

$$b = 2nA_s / (kd)^2 \cdot (.5h + a - kd)$$

$$= 2(15)32.45 / (14.6)^2 \cdot (15 + 4 - 14.6) = 20.15"$$

$$I = I_s + A_s (.5h + a - kd)^2 + b(kd)^3 / 3n$$

$$= 4270 + 32.45(15 + 4 - 14.6)^2 + 20.15(14.6)^3 / 3(15)$$

$$= 6,288$$

$$M = nf_o I / kd = 15(800)6288 / 14.6 = 5,160,000 \text{ in. lbs.}$$

$$V = A + B + 5w = 16,763 + 32,100 + 5(900) = 44,360\#$$

$$v = aV / nI \cdot (kd - .5a)$$

$$= 4(44,360) / 15(6,288) \cdot (14.6 - 4/2)$$

$$= 23 \# \text{ per sq. in.}$$

This section will be used as the moment and shear is within the allowable limits.

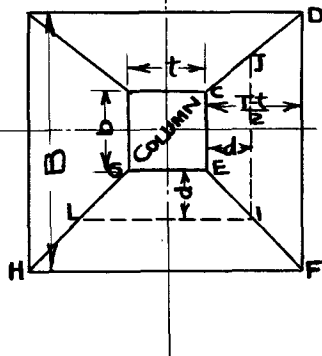
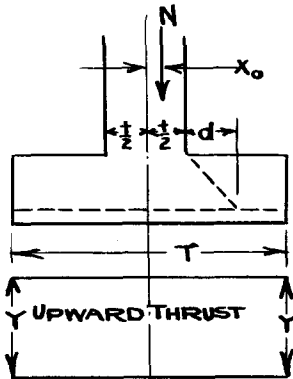
(Check for girder weight)

$$(34/12)(20.15/12)(150) = 800\# \text{ (900\# was assumed)}$$

(Column design computations are shown on the following print)

FOOTINGS

The load will be considered as acting at the center of the footing, the footing to be designed as square. Columns are 20 ft. long. Exterior column 14.5" square and interior column is 12" square.



L=5 Tons

$$f_c = .4(2000) = 800$$

$$f_s = 18000\#$$

$$v \text{ (allowable punching shear)} = .06f_c' = .06(2000) = 120$$

K=139 (from tables)

f_t = all. tensile stress concrete cannot exceed $.1(2000) = 200\#$, if so, steel must be used.

$$v = 40\#/\text{sq. in. allowable.}$$

DESIGN	EXT. COL.	INT. COL.
b in ft.	1.21	1.00
t in ft.	1.21	1.00
W (assumed wt. foot)	4500#	4000#
N (Load to column + length col. b. t)	217000#	174775#
BT = (N+W)/2000L	21.5 sq. '.	17.8 sq. '.
B in ft.	5.0	4.5
T (effective) in ft.	5.0	4.5
X_o in ft.	1.728	.338
T/2 in ft.	4.23	Use 4'
Y = N/BT in # sq. '.	8700	8600
$M_1 = (T-t)^2(2B+b)y/24$	37850' / #	28100' / #
$P_1 = (B+b)(T-t)(y/4)$	51200#	39250#
$M_2 = (B-b)^2(2T+t)y/24$	Same as M_1 (sq. col)	
$P_2 = (T+t)(B-b)(y/4)$	" " P_1 (" ")	
$d_1 = \sqrt{M_1/Kb}$ (mom)	15+3=18"	14+3.5=17.5"
$d_1 = P_1/12v_p$ (shear)	29+3=32"	27+3=30"
$f_t = 6M_1/bd^2$	183.5	31+
IJ (by graphics)	6.75'	6.5'
IL (" ")	6.75'	6.5'
V_1 (Area IJDF)Y	54810#	62300#
$v_1 = V_1/12(IJ)jd$	24# sq. "	31# sq. "
V_2 (ILFH)Y	Same as for V_1	
$v_2 = V_2/12(IL)jd$	" " " v_1	

COST OF ENCASED STEEL TYPE BRIDGE

Quantities will be computed for this bridge as they were for the reinforced concrete type bridge and the same unit prices will be used in estimating the final cost of the structure. Structural steel price, not quoted in the previous estimate, will be \$0.05 per pound set in place. The floor slab, hand railing and wearing surface cost will be the same as computed before and will be repeated here only in the final cost analysis. As stated before, the only excavation necessary is that necessary to place the four center footings.

(7 Interior Beam Stems)

Concrete - $2.25(1.26)128(7)/27$ - 94.0 cu. yds.

Steel - $112(128)7$ - 100,000#

(2 Exterior Beam Stems)

Concrete - $4(2.87)128(2)/27$ - 97.8 cu. yds.

Steel - $112(2)128$ - 28,700#

(3 Girders)

Concrete - $3(1.67)42.5(3)/27$ - 23.6 cu. yds.

Steel - $3(42.5)121$ - 15,400#

(6 Exterior Columns)

Concrete - $6(1.2)1.2(20)/27$ - 6.39 cu. yds.

Steel - $6(71)20$ - 8,520#

Re Steel - $30(4).167$ - 31#

(6 Interior Columns)

Concrete - $6(1.05)1.05(20)$ - 4.9 cu. yds.

Steel - $6(44)20$ - 5,280#

Re Steel - $30(4).167$ - 31#

(6 Exterior Column Footings)

Concrete - $5(5)2/27$ - 1.8 cu. yds.
 Steel - 75(5) - 375#

(6 Interior Column Footings)

Concrete - $4.25(4.5)2.25$ - 1.59 cu. yds.
 Steel - 375#

(Excavation)

Exterior Footings - $5(5)2/27$ - 1.8 cu. yds.
 Interior Footings - $2(4.5)4.25/27$ - 1.6 cu. yds.

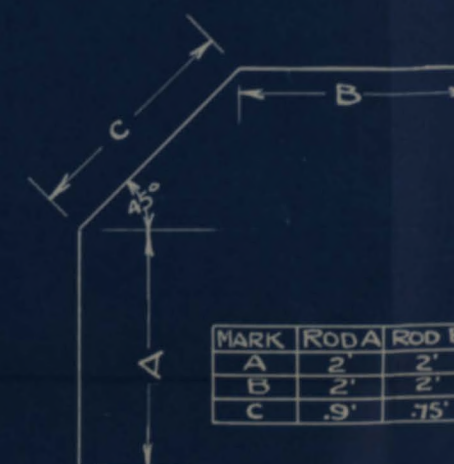
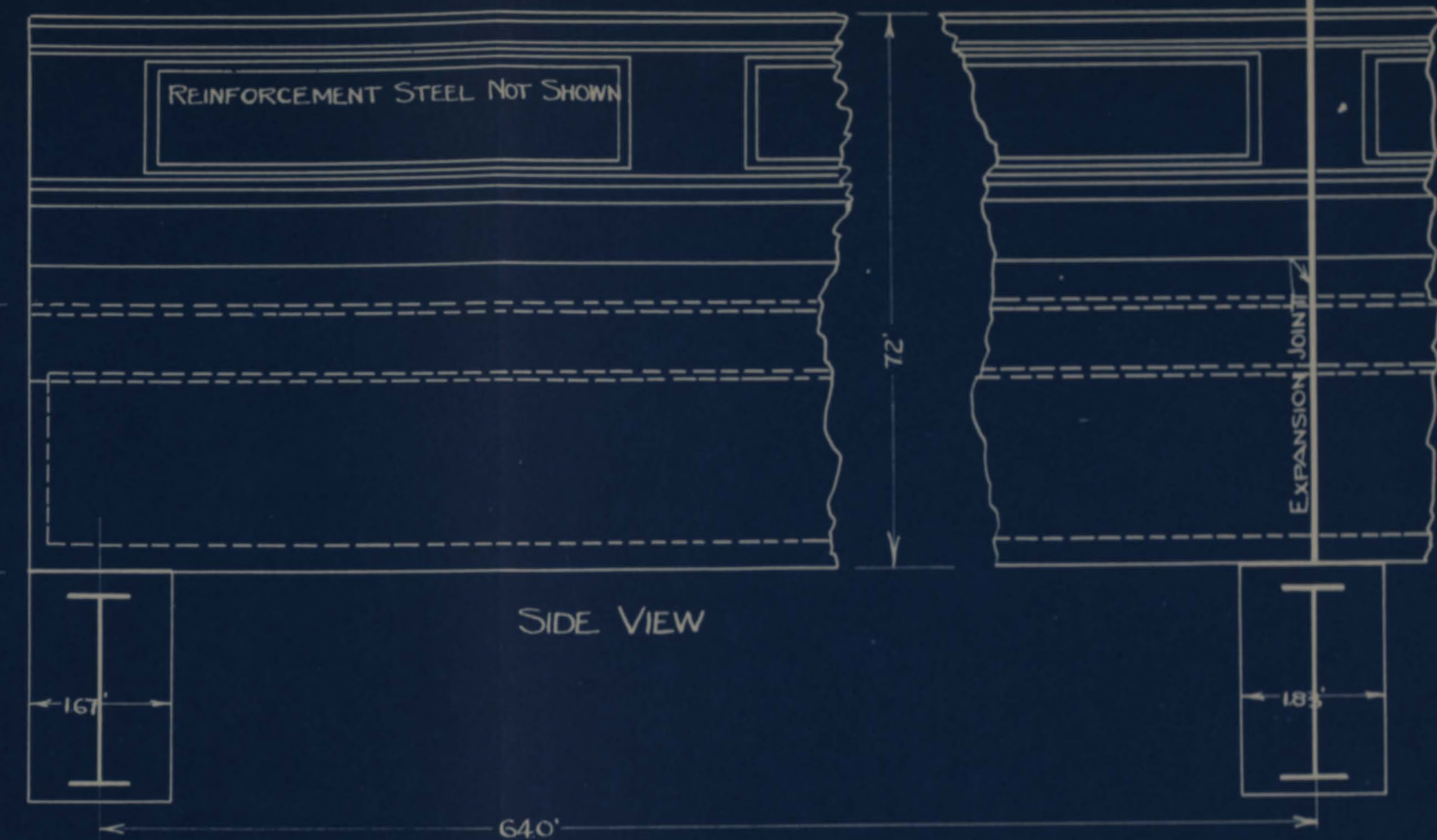
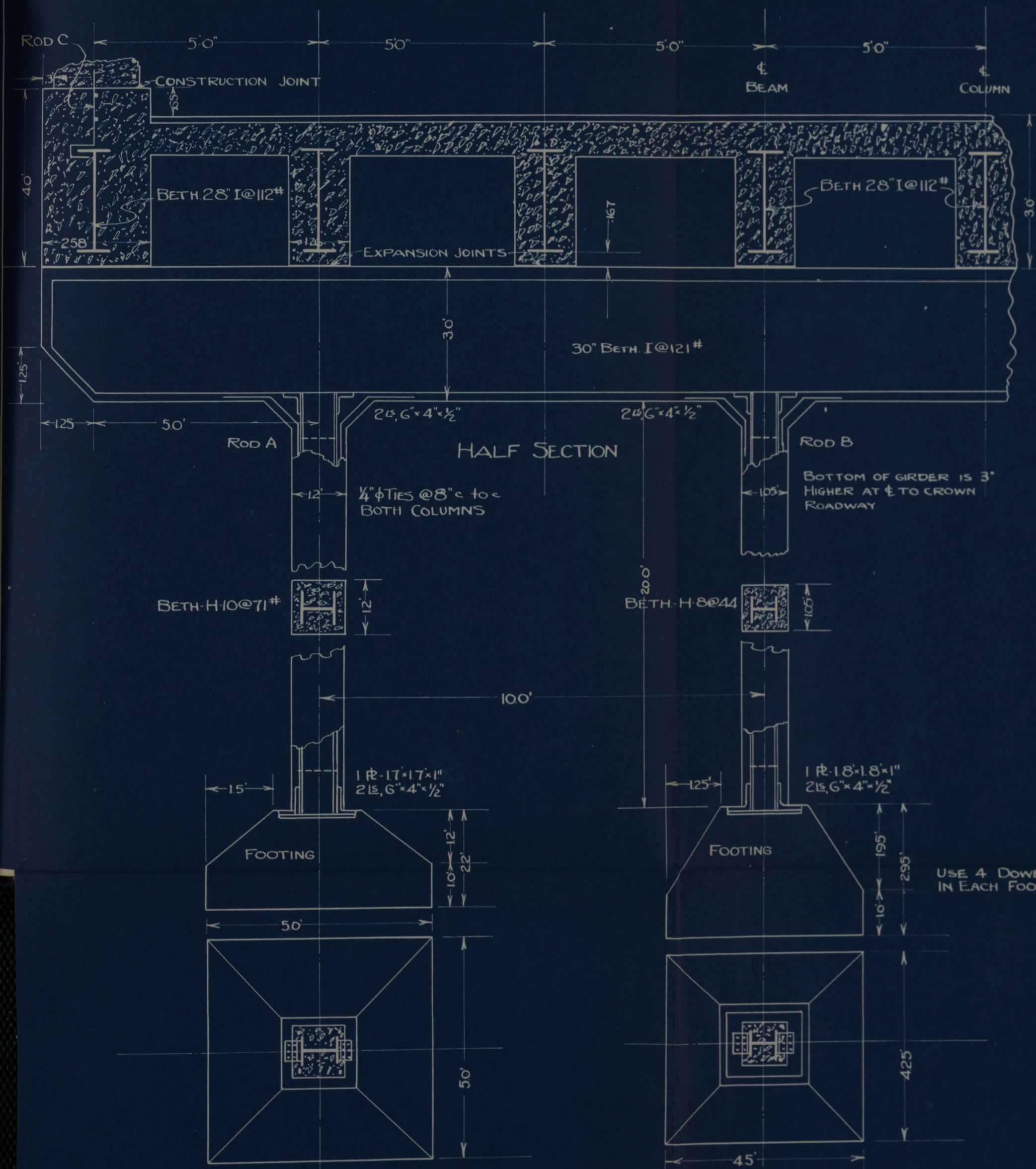
(Forms)

The structural steel sections will be bolted in place as a rigid frame structure, the forms being hung from these sections after they are in place. Cost of forms will then be practically the same as for the reinforced concrete bridge minus the intermediate bracing needed to hold the forms in place on that bridge.

(Totals and Cost)

Concrete	- 380 - .07(380) cu. yds. at \$20.00 (It is estimated that 7% of the structure is occupied by the steel and this is deducted from the total concrete).....	= \$7073.80
Structural Steel	- 174,497# at \$0.05.....	= 8724.85
Forms	- 11,500 sq. ft. at \$0.25.....	= 2875.00
Excavation	- 4 cu. yds. at \$1.00.....	= 4.00
Wearing Surface	- 540 sq. yds. at \$0.60.....	= 324.00
Re Steel	- 15,755# (Included in structural steel--use as at \$0.02 here)...	= 315.10
Total		= \$19,316.75

NOTE-
THE SAME HAND RAILING AND FLOOR SYSTEM AS DESIGNED FOR THE
REINFORCED CONCRETE BRIDGE WILL BE USED HERE



MARK	ROD A	ROD B	NO REQ'D
A	2'	2'	36 1/2" Rod A
B	2'	2'	36 1/2" Rod B
C	.9'	.75'	

ROD C-128 REQ'D

GA. SCHOOL OF TECHNOLOGY
ENCASED STEEL SLAB, BEAM, GIRDER, COLUMN BRIDGE

DESIGNED BY-R.H.Y.
DRAWN " - "
TRACED " - "
REVISIONS

SCALE-1/2"=1'
DATE-2-27-33
DRAWING NO-2-C

THROUGH PLATE GIRDER BRIDGE

Span - 2 at 64' - 128'

GENERAL DESCRIPTION OF BRIDGE

This is to be a 64 ft. through plate girder bridge with a 40 ft. roadway(c to c of plates). Floor shall be reinforced concrete resting directly on closely spaced floorbeams, no stringers to be used. Wearing surface of 30# per. ft. of roadway shall be used. Bridge shall be designed to comply with the General Specifications for Steel Highway Bridges.

LOADS

(Dead Load)- The dead load shall consist of the weight of the girders, floorbeams, floor slab and wearing surface. Reinforced concrete assumed at 150# per cu. ft.

(Live Load)- The bridge shall be designed for a 12 ton truck loading, 19,200# on two rear wheels(9,600# on each rear wheel) and 4,800(2,400# each) on front wheels. Live load for girders, as taken from specifications, is to be 83# per. sq. ft. of roadway.

(Impact)- 30% shall be allowed for impact.

(Wind Load)- No appreciable effect on this type bridge.

GENERAL DIMENSIONS

Span- About 64 ft. c to c of bearings.

Width roadway- About 38 ft.

Spacing of girders- About 40 ft. c to c.

Depth of girder- At least $(1/12)(64)(12) = 64$ inches. A depth of 66 inches back to back of flange angles will be used.

Rivets- 3/4" diameter rivets will be used throughout.

DETAILED DIMENSIONS

Gusset plates will be used to provide lateral support for the girders. These plates shall come at the floorbeams and they

shall be attached to the stiffeners. Web plate thickness shall be a minimum of $66/160 = .41"$. $1/2"$ plates shall be used. Stiffeners must be used since $(1/16)(64) = 1.07"$ is greater than web thickness. Stiffeners shall be placed at every floorbeam (5')

DESIGN OF FLOOR SLAB

Stringers shall be omitted. Assumed thickness of slab 8".

Total dead load is:

Slab weight $(8/12)(150) = 100\#$ per. sq. ft.

Wearing surface $= 25\#$ " " "

Total $= 125\#$ " " "

$M_{(D)max.} = (1/14)wl^2 = (1/14)(125)(5)(12) = 2,680$ in. lbs.

$M_{(D)max. neg.} = -(1/9.5)(125)(5)(12) = -3,950$ in. lbs.

M due to wheel loads = .2096PL

$= (1/5)(9,600)(5)(12) = 115,200$ in. lbs.

$M_{(neg.)}$ due to wheel loads = $-(1/7.7)PL$

$= -(1/7.7)(9,600)(5)(12) = -74,800"/\#$

$M_{L(max)} = 115,200 + .30(115,200) = 209,000$ in. lbs. occurring at center of end panel.

neg. $M_{L(max)} = -74,800 + .30(74,800) = -103,300$ in. lbs. occurring at first transverse beam.

Distribution- $E = .7(5) + 1.25 = 4.75'$ (outer zone)

$E = .5(4.75 + 3) = 3.87'$ (center zone)

The maximum negative moment due to live load and impact on a longitudinal strip 1' wide at the center of the roadway is $-103,300/3.87 = -26,700$ in. lbs. Adding moment due to dead load, $(26,700 + 3,950) = -30,650$ in. lbs. which is the maximum

negative moment at the center of the roadway over the first intermediate transverse beam.

Assuming $d = 6.5"$, $j = 7/8"$, $k = 0.38$, $jd = 5.69$,

$kd = 2.67$. $F = M/jd = 30,650/5.69 = 5,380\#$.

$A_s(\text{top of slab}) = 5,380/16,000 = .33 \text{ sq. in. per. ft. of width}$, and $.33/12 = .027 \text{ sq. in. per. in. of width}$.

Use $1/2"$ ϕ rods at $6"$ giving $.39 \text{ sq. in. per. ft. of width}$.

$f_c = 2F/kdb = 2(5380)/(2.67(12)) = 335\#$ compression at bottom of concrete. Allowable is $650\#$

$M_L(\text{at center of end strip } 1' \text{ wide}) = 209,000/3.87 = 55,400"/\#$

$M_L(\text{total at center}) = 55,400 + 2,680 = 58,080 \text{ in. lbs.}$

$F = M/jd = 58,080/5.69 = 10,200\#$

$A_s(\text{bottom of slab}) = 10,200/16,000 = .638 \text{ sq. in. per ft. of width}$. Use $3/4"$ ϕ bars at $6"$ giving $.88 \text{ sq. in. per ft. width}$. This arrangement will be used in the bottom of the slab.

$f_c = 2F/(kd)b = 2(10,200)/(2.67(12)) = 630\#(\text{allowable } 650)$

(Design of slab for outer zone) - Considering a longitudinal strip at the first intermediate transverse support from end of bridge, strip being considered $1'$ wide:

$-M_L(\text{max}) = (103,300/4.75) + 3,950 = 25,700 \text{ in. lbs.}$

Assume $d = 5.5$, $j = .875$, $k = .38$, $jd = 4.81$, $kd = 2.09$

$F = M/jd = 25,700/4.81 = 5,340\#$

$A_s(\text{in top}) = 5,340/16,000 = .332 \text{ sq. in. per. ft. width}$

Use $1/2"$ ϕ bars at $6"$ giving $.38 \text{ sq. in. per. ft.}$

$f_c = 2(5,340)/2.09(12) = 214\#$ (allowable is $650\#$)

Next, considering positive moment at the same place,

$$M_L = (209,000/4.75) + 2,680 = 46,800 \text{ in. lbs.}$$

$$F = 46,800/5 = 9,360\#$$

$A_s = 9,360/16,000 = .583 \text{ sq. in. per. ft. width.}$ Use
5/8" ϕ rods at 6" giving .61 sq. in. area per. ft.
width.

$$f_o = 2(9,360)/2.09(12) = 750\#(\text{slightly in excess of the allowable but will be used.})$$

DESIGN OF FLOORBEAMS

Average thickness of floor slab including wearing surface is 7.5". Assumed weight of floorbeams = 150# per. ft. length.

Dead load per. ft. of beam =

$$\text{From slab } (7.5/12)(5)(150) = 937\# \text{ per. ft.}$$

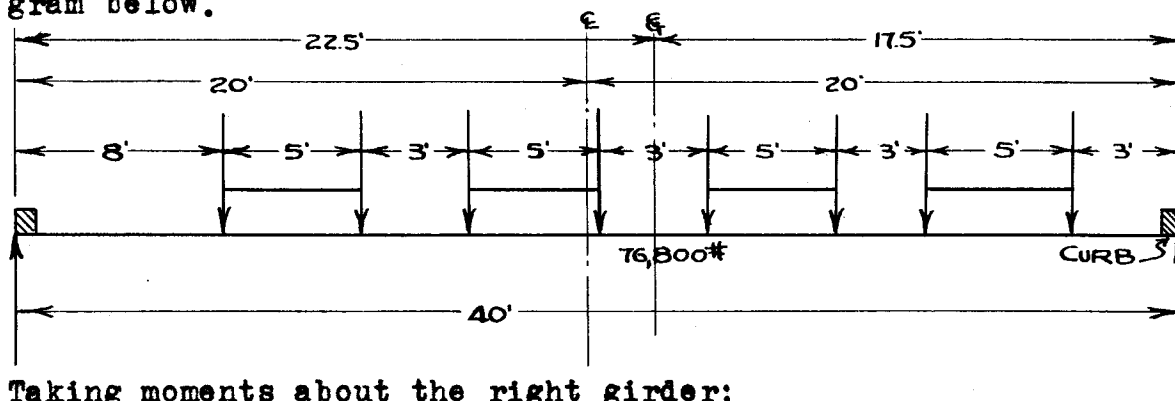
$$\text{From beam} = 150\# \text{ " " }$$

Each floorbeam length is 40 ft.

$$M_D = (1/8)(1,087)(40^2)(12) = 2,610,000 \text{ in. lbs.}$$

$$S = 2,610,000/18,000 = 145 \text{ in.}^3 \text{ required for dead load}$$

The maximum moment due to live load occurs when the rear wheels of four trucks are on the bridge as shown in the diagram below.



Taking moments about the right girder:

$$R = (76,800)17.5/40 = 33,600\# \text{ for } R_L$$

$$R = (76,800)22.5/40 = 43,200\# \text{ for } R_R$$

Taking moments about the wheel nearest the center of the beam

and considering forces to the right:

$$\begin{aligned} M_L(\max) &= 12 (43,200)19 - (9,600)3 + (9,600)8 \\ &\quad + (9,600)11 + (9,600)16 \\ &= 5,529,600 \text{ in. lbs.} \end{aligned}$$

$$\text{Impact} = .30(5,529,600) = 1,661,600 \text{ in. lbs.}$$

$$M_L(\text{total}) = 5,529,600 + 1,661,600 = 7,191,200 \text{ in. lbs.}$$

$$S = 7,191,200/18,000 = 400 \text{ in.}^3 \text{ for live load.}$$

$$\text{Total section modulus} = S = 145 + 400 = 545 \text{ in.}^3$$

A Bethlehem G-30 at 180.0# and with an S = 556.2 will be used for all interior floor beams.

Maximum reaction(end shear) =

$$\text{Dead Load } (1,087)20 = 21,740$$

$$\text{Live Load} = 43,200$$

$$\begin{array}{rcl} \text{Impact} & & = 13,100 \\ \text{Total} & = & \underline{78,040\#} \end{array}$$

(Design of End Floorbeams)

$$\text{Dead Load on end floorbeams} = (2/3)1087-725\# \text{ per. ft.}$$

$$M_D = (1/8)(725)(40^2)(12) = 1,740,000 \text{ in. lbs.}$$

$$S = 1,740,000/18,000 = 96.6 \text{ in.}^3$$

$$\text{Total } S = 400 + 97 = 497 \text{ in.}^3$$

A Bethlehem G-30 at 173# will be used for the end floorbeams

DESIGN OF MAIN GIRDERS

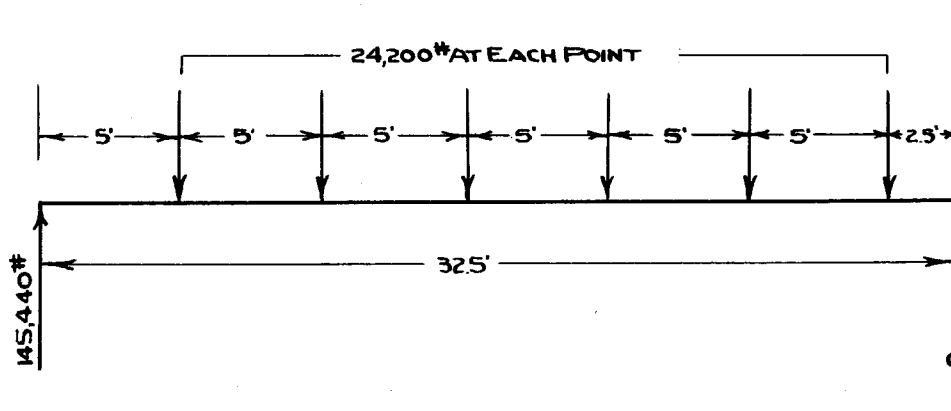
All loads are applied to the girders through the floorbeams at the panel points (except for the weight of the girders).

Dead load at panel points-

$$\text{From floorbeams} = (1,087)20 = 21,740$$

$$\begin{array}{rcl} \text{From main girders(assumed)} & 500(5) & = 2,500 \\ \text{Total} & = & \underline{24,240\#} \end{array}$$

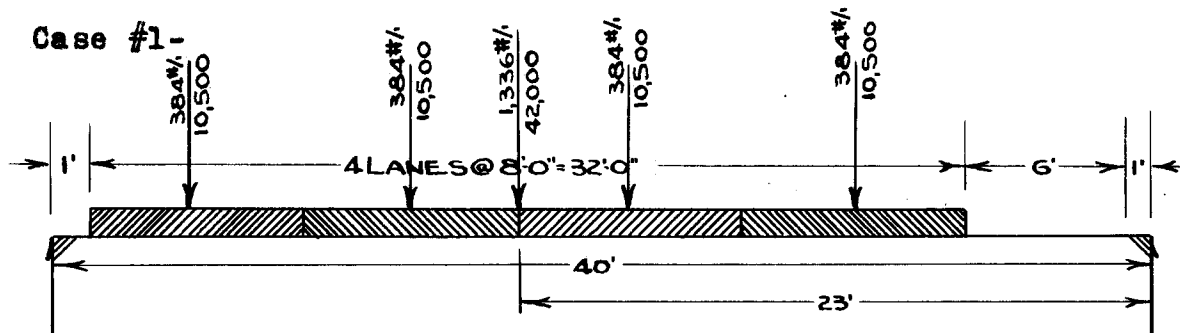
A diagram of this loading is shown on the next page top.



$$M_D = [(145,440)(32.5) - 6(24,240)16.25] / 2 = 28,320,000 \text{ in.}/\text{ft}$$

The live load consists of a uniform load and of a single concentration. To obtain concentration, two cases shall be considered, the one producing the greater moment to be used.

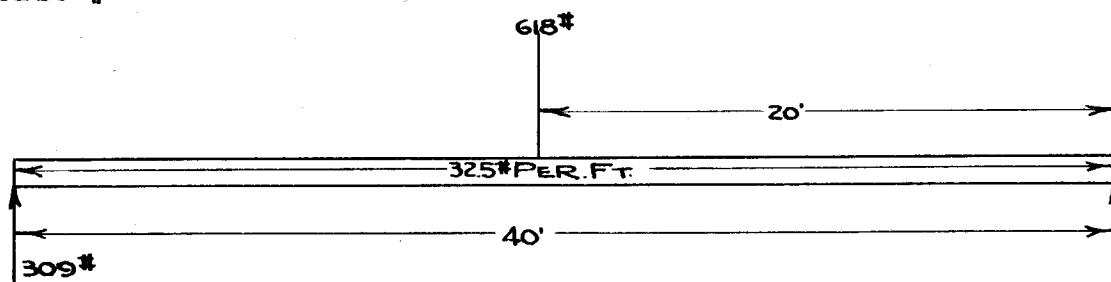
Case #1-



Here, the uniform load of 384#(h-12) per. ft. is considered to be on 4 traffic lanes, and

$$R_L = (1,336)23/40 = 768\#$$

Case #2-



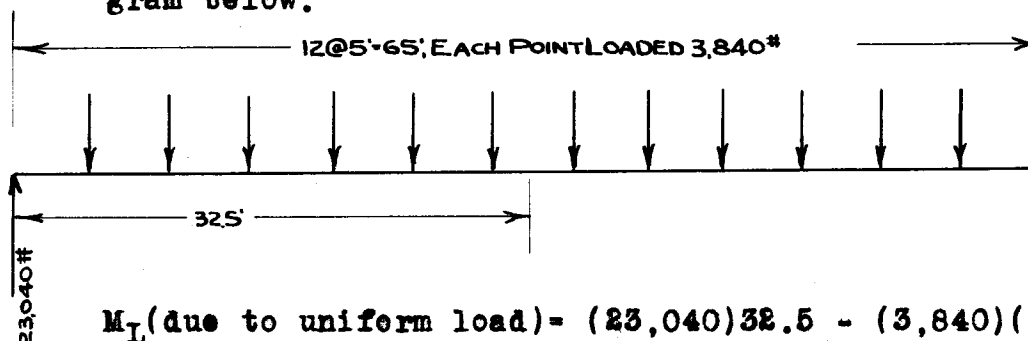
Here, the uniform load is reduced 22% and distributed over the entire roadway. The load per. ft. is, therefore,

$$(384/8) - .22(384/8) = 32.5\# \text{ per.ft.}$$

$$R_L = (618)20/40 = 309\#$$

Case #1 shall rule, then the panel load for uniform load is:

$(768)(5) = 3,840\#$. This loading is shown in the diagram below.



$$M_L(\text{due to uniform load}) = (23,040)(32.5) - (3,840)(6)(16.3) \\ = 4,510,000 \text{ in. lbs.}$$

The 10,500# concentrated loads (given for H-12 loading) will be placed as indicated by case #1 on the preceding page. The single concentrated load on the girder will be:

$$(42,000)23/40 = 24,100\#$$

Placing this load at the center, the maximum moment due to live load is:

$$(24,100/2)(35)(12) = 5,060,000 \text{ in. lbs.}$$

$$\text{Total } M_L = 4,510,000 + 5,060,000 = 9,570,000 \text{ in. lbs.}$$

$$\text{Impact} = 50/(65+125) = .26(9,570,000) = 2,520,000 \text{ in. lbs}$$

$$M_L + M_I = 12,090,000 \text{ in. lbs.}$$

$$M_L + M_I + M_D = 12,090,000 + 28,320,000 = 40,410,000 \text{ in. lbs.}$$

Economic depth of girder is

$$x = 1.055\sqrt{M/ft}, \text{ t-thickness of web.}$$

$$= 1.055\sqrt{40,410,000/18,000(.5)} = 70.05"$$

A 72" by 1/2" plate, back to back of flange angles, will be used
Assume the effective depth to be 70". Flange area required is

$$A_s = M/af_s = 40,410,000/(70)18,000 = 31.8 \text{ sq. in. less}$$

$$1/8 \text{ web} = 70(.5)1/8 = 4.4 \text{ sq. in.}$$

$$A_s = 31.8 - 4.4 = 27.4 \text{ sq. in.}$$

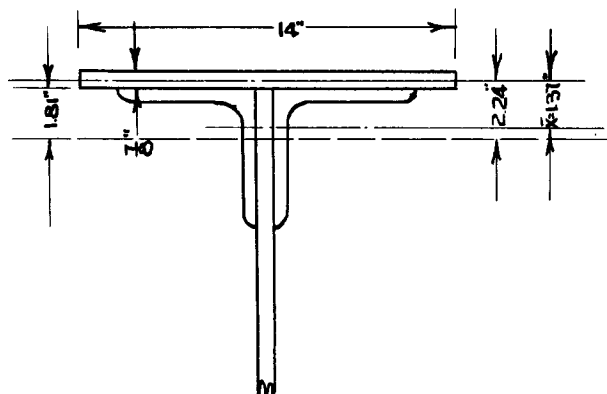
The following sections will be used in the flange:

2 angles 6" by 6" by 7/8" = 19.46 - 2(.77) = 17.92 sq. in.

1 cover plate, 14" by 7/8" = 12.25 - 2(.77) = 10.71 " "

Totals = 31.71 28.63 " "

Check to determine if recalculation of flange area is necessary:



Taking moments about the center of the cover plate:

$$x(31.71) = 19.46(2.24), x = 1.37$$

Subtracting 1/2 thickness of cover plate, then

$$d = 72 - 2(1.37 - .43) = 70.12" \text{ (only .12" over assumed } d)$$

Maximum end shear occurs when all panels are loaded with the uniform live load, and the concentrated load. Referring to the figure below and using 15,600#(given for H-12 loading) for shear, then the live load shear is:

$$V_L = 4(13,500)(23/40) = 31,050\#$$

Total end shear:

$$V_{(Total)} = V_L + 3840(6) + 31,050(11/12) \text{ (note-1/12 to ends)} \\ = 51,600\#$$

$$V_I = (50/90)51,600 = 13,600\#$$

$$V_D = 24,240(6) = 145,400\#$$

$$V_{(Total)} = 51,600 + 13,600 + 145,400 = 210,640\#$$

Pitch of the rivets is obtained from the formula $p = rh / (S(1 + A'/8A))$

S = shear on girder at any point.

A = cross section area of one flange.

A' = " " " " " web.

f = flange increment.

r = pressure exerted on flange by each rivet.

h = vertical distance between rivets in the two flanges.

p = spacing.

Substituting in the formula:

$$p = 9,000(66)/210,640(1 + .5(72)/((8)(28.63)))$$

= 3.29". 3" will be used for spacing in the vertical legs of the flanges.

Maximum reaction on the end = dead load + uniform live load over the entire span + concentrated live load.

$$R = 6.5(24,240) + 65(768) + 31,050 = 232,550\#$$

Bearing area required in end stiffeners =

$$232,550/18,000 = 12.91 \text{ sq. in.}$$

4 angles, 5" by 4" by 11/16 gives an effective area of 13.7 sq.".

The intermediate stiffeners will be made up of angles 5" by 4" by 5/8", placed in pairs at 5'-0".

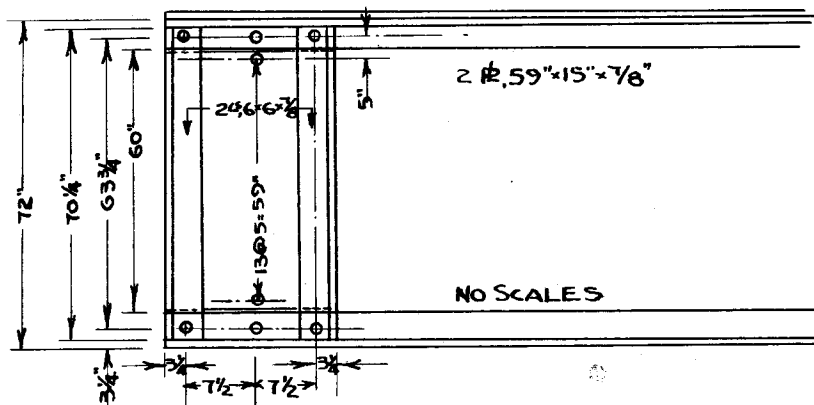
Area of bearing on masonry plates:

$$A = V/f = 232,550/600 = 387 \text{ sq. in.}$$

DETAILS

Rivets required in end stiffeners = $232,550/9,000 = 26$

Details of the end stiffeners are shown below:



(Details of Web Splice)- Each girder will be spliced at about the third point, at about 22' from the end of the girder, and at stiffeners.

Dead load for bridge-

14 floorbeams 14(180)40.....100,800#

Slab and wearing surface.....238,000

Girders(assumed)..... 38,000

Total....376,800#

Dead load per. ft. of girder is:

$$376,800/65(2) = 2,880\#$$

Live load per. ft. of girder(using 125# per. ft. and including impact) is:

$$1.30(125)40/2 = 3,250\#$$

Dead load shear at the point of splice is:

$$V_D = 2,880(64/2) = 2,880(22) = 28,800\#$$

$$\begin{aligned} M_D(\text{at splice}) &= (1/2)(2,880)(65)(22) - (1/2)(2,880)(22^2) \\ &= 1,362,000 \text{ ft. lbs.} \end{aligned}$$

The maximum live load shear will occur with the uniform load covering the girder up to the point of splice, and is:

$$V_L = (3,250)(43)(22)/65 = 47,300\#$$

The maximum live load moment occurs with the entire span loaded, and is:

$$\begin{aligned} M_L &= (3,250)(65)(22)(1/2) - (1/2)(3,250)(22^2) \\ &= 534,000 \text{ ft. lbs.} \end{aligned}$$

The total moment at the section is:

$$M_{\text{(Total)}} = 1,362,000 + 534,000 = 1,896,000 \text{ ft. lbs.}$$

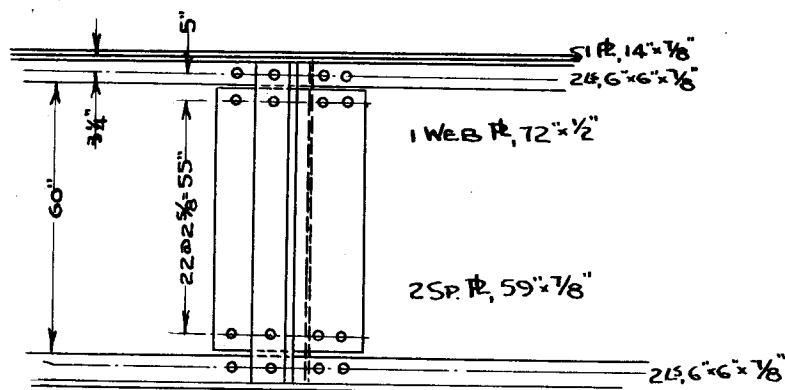
The web carries this ratio of moment:

$$(\text{Web area as flange}/\text{Area of one flange})(M)$$

$$M_{(\text{By Web})} = (35(1/8)/28.63)(1,896,000) \\ = 290,000 \text{ ft. lbs.}$$

$$V_{(\text{Total})} = 47,300 + 28,800 = 76,100\#$$

Details of web splice are shown below:



The above arrangement will be investigated to determine whether satisfactory or not. The stress in the outermost rivet is given by the formula:

$$r = \sqrt{(V/2n)^2 + (Mdn/2 \sum d^2)^2}, \text{ where}$$

$2n$ = number of rivets on one side of web splice.

M = moment carried by the web.

dn = distance back of flange angle to neutral axis.

$\sum d^2$ = summations of spacings

$$= 2(25/8)^2(1^2 + 2^2 + 3^2 + 4^2 + 5^2 + 6^2 + 7^2 + 8^2 + 9^2 + 10^2 + 11^2)$$

$$= 6,940 \text{ in.}^2 = 578 \text{ ft.}^2$$

$$r = \sqrt{(76,100/44)^2 + (290,000(27)/1,156)^2}$$

= 7,000#. The allowable bearing of a 3/4" rivet on a .5" plate is 9,000#. This splice proves satisfactory and will be used.

(Design of End Bearings) - Roller bearings are not required for spans less than 70' so sliding bearings will be used for this girder. Slotted holes will be used in the sole plates to allow for a movement of at least 5/8" at one end. The area of the

wall plate must be at least:

$$A = V/f = (V_D + V_L)/f = (28,800 + 47,300)/600 = 127 \text{ sq.}^{\prime\prime}.$$

COST OF PLATE GIRDER BRIDGE

The span as used for this investigation necessitates the use of very large steel members. For this reason it is possible, since there is less fabrication, that the price of structural steel as quoted throughout this Thesis may be in excess. Gravity piers will be used for the girder and truss bridge. These piers are not designed here, but the quantity of concrete necessary can be determined from the curves as found on pp. 340 of Kirkham's "Highway Bridges". The weight estimate of the plate girder bridge is given below.

No.	pcs.	Shape	Section	Length Ft. in.		Wt. per. ft. lb.	Weight each	Total
2		Cover Pl.	14 x 7/8	64	3	41.25	2,650	5,300#
4		Angles	6x6x7/8	64	0	9.73	623	2,492
2		Web Pl.	6'x1/2"	23	10	121.68	2,900	5,800
16		Angles	5x4x9/16	5	10	4.75	28	448
2		Web Pl.	6'x1/2"	40	2	121.68	4,889	9,778
22		Stf. Ang.	5x4x1/2"	5	10	4.75	28	616
22		"	"	3	2	4.75	15	330
8		Fill. Pl.	4'11x7/8	1	3	174.41	218	1,774
22		"	"	0	4	174.41	58	1,276
22		"	4'4x7/8"	2	8	174.41	471	10,362
11		G-Beam	G-30	40	0	180.	7,200	79,200
2		"	"	40	0	173	6,920	13,840
52		Angles	6x6x7/8	2	1	9.73	21	1,092
2		"	"	64	0	9.73	623	1,246
Total								133,524#

Details, including rivet heads, gussets, etc. 5% Total= 6,676#

Total estimated weight of steel in place - 140,200# - 70 Tons
 Quantity of concrete required for piers (from curves) - 60 cu. yds.

FINAL COST

140,200# structural steel in place at \$0.05	- \$7,010
46.2 cu. yd. concrete for floor slab at \$20.00	- 924
273 yd. wearing surface in place at \$0.60	- <u>164</u>
Total cost of one span	- \$8,098
Two spans required	- 16,196
3 piers in place 60 x 3 x \$20.00	- <u>3,600</u>
Total cost of bridge	- \$19,796

HIGH RIVETED PRATT TRUSS BRIDGE

Span - 128 ft.

GENERAL DESCRIPTION

This is to be a through bridge having riveted Pratt trusses with parallel chords. The floor is to be composed of a reinforced concrete slab, with additional wearing surface placed on I-Beam joists.

LOADS

(Dead Load) The dead load consists of the weight of the reinforced concrete floor slab at 150# per cu. ft., the joists, the floorbeams, trusses and lateral bracing.

(Live Load) The bridge will be designed for loading of a 12-Ton truck concentrated load or a uniform load of 85# per sq. ft. of roadway for the floor and its supports. The live load for the trusses is given in specifications as 73# per sq. ft. of roadway.

(Impact) 30% of live load for floor and its supports, and of $100/(L+300) = 100/(128+300) = 23.4\%$ for the truss members.

(Wind Load) The specifications require that the lower lateral bracing be designed for a moving wind load of 300# per. ft. of bridge, and the upper lateral bracing be designed for a moving wind load of 150# per. ft. of bridge.

DIMENSIONS

Span, 128 ft.(about) c to c end bearings, panel length will be 18'-01" c to c.

DEPTH OF TRUSSES

The trusses must have a depth sufficient to provide head room of 15' for a width on center line of bridge of 20'. The bottom of the floorbeam will be placed about even with the

bottom of the lower chord. The floorbeam will probably have a depth of about 30" and the slab a depth of about 7". The top of the joists will be even with the top of the floorbeam. Using a depth of 22' c to c of chords the various parts will occupy the following depths:

Headroom.....15'(about)
 Floorbeam..... 2'-06"
 Slab..... 0'-07"
 Portal..... 3'-00"
 Total.....22'- (about)

DESIGN OF FLOOR SYSTEM

(Floor Slab) Stringers will be spaced at 3', the assumed depth of floor slab is 7". Dead load per sq. ft. floor is:

Slab.....(7/12)150 = 87.5#
 Extra Floor Covering..... = 25 #
 Total = 112#

$$M_D(\text{positive}) = (1/14)wl^2 = (1/14)112(3)^2 12 = 863 \text{ in. lbs.}$$

$$M_D(\text{negative}) = -(1/9)wl^2 = -(1/9)112(3)^2 12 = -1275 \text{ in. lbs.}$$

Heaviest rear wheel for live load moment is 9600#

$$M_L(\text{positive}) = (1/5)9600(3)12 = 69,000 \text{ in. lbs.}$$

$$M_L(\text{negative}) = -(1/6)9600(3)12 = -57,600 \text{ in. lbs.}$$

Note-The above formulas are not exact, but near enough so for all practical purposes.

$$E(\text{effective width to carry wheel load}) = 0.7(2D + W)$$

$$= 0.7(2 \cdot 1.5 + 1.25)$$

$$= 2.97'$$

The maximum moment on a strip 1' wide is $69,000/2.97 = 23,600"/\#$

$$\text{Impact} = 50/(3+125)23,600 = 9250 \text{ in. lbs.}$$

Total positive moment on a ft. wide strip is

$$L = 23,600$$

$$I = 9,250$$

$$D = \frac{863}{33,713}$$

T = 33,713 in. lbs. which occurs in center of outer panel.

$$\text{Negative } M = -57,600/2.97 = -19,400 \text{ in. lbs.}$$

$$I = 50/(3+125)19,400 = -7,680 \text{ in. lbs.}$$

Total negative moment is:

$$L = -23,600$$

$$I = -7,680$$

$$D = -1.275$$

T = -32,555 in. lbs., which occurs over an intermediate support.

$$\text{Assume } d = 5", j = 7/8, k = .38, jd = 4.35, kd = 1.90$$

Considering negative moment over the support first:

$$F = M/jd = 32,555/4.35 = 7,470 \# \text{ stress in top steel}$$

$$A_s = 7470/18000 = .415 \text{ sq. in. per. ft. width of slab.}$$

$$.415/12 = .035 \text{ sq. in. per. in. width of slab.}$$

A 5/8" ϕ bar has an area of .3068 sq. in.

$$.3068/.035 = 8.7" (\text{use } 8.5")$$

$$f_c = 2F/kdb = 2(7470)/1.90(12) = 660 \#/\text{sq. in. compression}$$

at bottom of slab. This exceeds the allowable by only 10# so the design will not be changed.

Next, considering positive moment:

$$F = 33,713/4.35 = 7700 \#$$

$$A_s = 7700/18000 = .427 \text{ sq. in. per. ft. width slab.}$$

$$A_s = .427/12 = .035 \text{ sq. in. per. in. width slab.}$$

Using 5/8" ϕ bars, .3068/.035 = 8.75" spacing bottom slab.

$$f_c = 2(7700)/1.90(12) = 680 \#/\text{sq. in. compression at top of}$$

slab, which is only slightly in excess of the allowable.

The slab as designed is satisfactory. The total depth of the slab shall be $5 + 1\frac{1}{2} = 6\frac{1}{2}"$. The slab shall be made $6\frac{3}{4}"$ at curb and $7\frac{3}{4}"$ at crown, this including wearing surface and crowning.

(Stringers) Stringers shall be 18' long and spaced 3' c to c.

Dead load weight per ft. of stringer is:

$$\text{Floor} = 7/12(150)3 = 262\#$$

$$\text{Beam(assumed)} = 30\#$$

$$\text{Total} = 292\# \text{ per. ft. of beam}$$

$$M_D = (1/8)wl^2 = (1/8)292(18)^2 \cdot 12 = 142,000 \text{ in. lbs.}$$

Maximum live load moment occurs when the large wheel is at mid span, and is (not considering distribution)

$$(9600)/2 \cdot 9 \cdot 12 = 518,000 \text{ in. lbs.}$$

The distribution factor is $3/4.5 = .66$ (4.5 is for two lane traffic, 6 is for 1 lane traffic. This bridge is for four lane traffic)

$$M_L = 518,000/.66 = 342,000 \text{ in. lbs.}$$

$$I = 50/(18+125) \cdot 342,000 = 119,800 \text{ in. lbs.}$$

$$M_{(\text{Total})} = 142,000 + 342,000 + 119,800 = 603,800 \text{ in. lbs.}$$

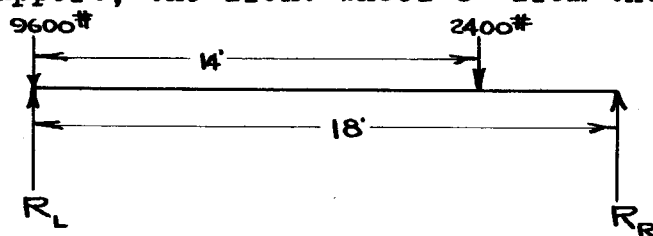
$$S = M/f_s = 603,800/18,000 = 33.5 \text{ in.}^3$$

A Bethlehem 12" by 28.0# I will be used.

Maximum end shear due to dead load is:

$$V = (262 + 28)8 = 2,320\#$$

The maximum live load shear occurs when the large wheel is over the support, the front wheel 4' from the other support.



$$R_L = [(2400 \cdot 4) + (9600 \cdot 18)] / 18 = 10,100\#$$

$$I = .35(10,100) = 3,550\#$$

$$V_{(Total)} = 2,320 + 10,100 + 3,550 = 15,970\#$$

(Floorbeams) The length of floorbeams will be considered as 40' c to c trusses. Dead load per. ft. of an interior beam:

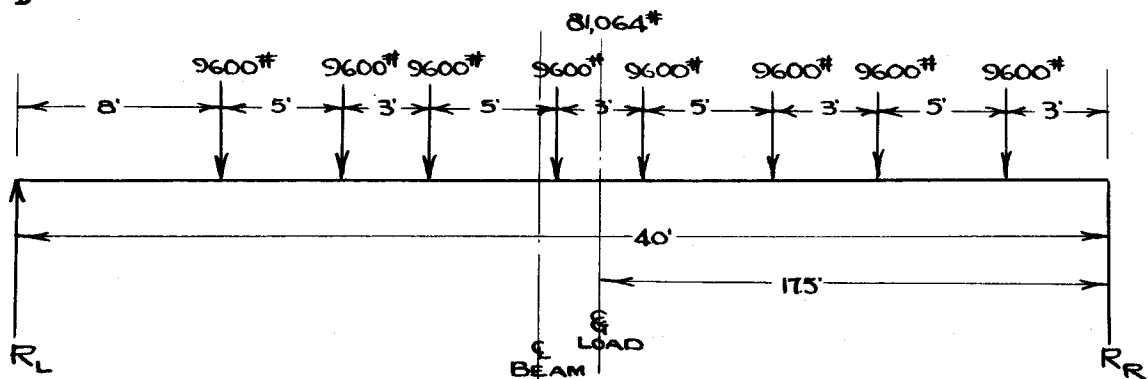
$$\text{Floor} \dots (7/12)150(18) = 1,575\#$$

$$\text{Stringers} \dots (9 \cdot 28 \cdot 18) / 40 = 113$$

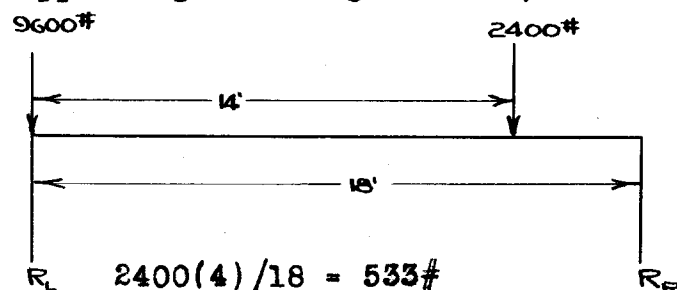
$$\text{Beam(assumed)} \dots = \frac{150}{40}$$

$$\text{Total} = 1,838\# \text{ per. ft. of beam}$$

$$M_D = (1/8)wl^2 = (1/8)1838(40)^2 / 12 = 4,510,000 \text{ in. lbs.}$$



The maximum live load moment will occur when the large wheels are over the beam and in the position as shown above. The front wheels, 2400# each, are 4' from the adjacent beam. Part of this front wheel load will come through the stringers to the beam supporting the large wheels, as shown below. For



the convenience of computations, this 533# will be added to the 9600#, making the total load = 9600 + 533 = 10,133#.

Taking moments about the adjacent beams:

$$R_L = 81,064(17.5)/40 = 35,500\#$$

$$R_R = 81,064(22.5)/40 = 45,500\#$$

Taking moments about the wheel nearest the center line of the beam and considering forces to the right:

$$M_L = [(45,500 \cdot 19) - 10,133(3 + 8 + 11 + 16)]12 = 5,640,000"/\#$$

$$I = 50/(40+125) \cdot 5,640,000 = 1,710,000 \text{ }"/\#$$

$$M_{(Total)} = 4,510,000 + 5,640,000 + 1,710,000 = 11,860,000"/\#$$

$$S = M/f_s = 11,860,000/18,000 = 612 \text{ in.}^3$$

This requires a Bethlehem B-30 at 200# per. ft.

End shear due to dead load:

$$V = (1838 + 50)20 = 37,800\#$$

$$I = .30(37,800) = 11,340\#$$

$$V_{(Total)} = 37,800 + 11,340 = 49,140\#$$

End Beam Design -

$$M = (2/3)4,510,000 + (5,640,000 + 1,710,000) \\ = 10,350,000 \text{ in. lbs.}$$

$$S = M/f_s = 10,350,000/18,000 = 575$$

A B-30 at 190# per. ft. will be used.

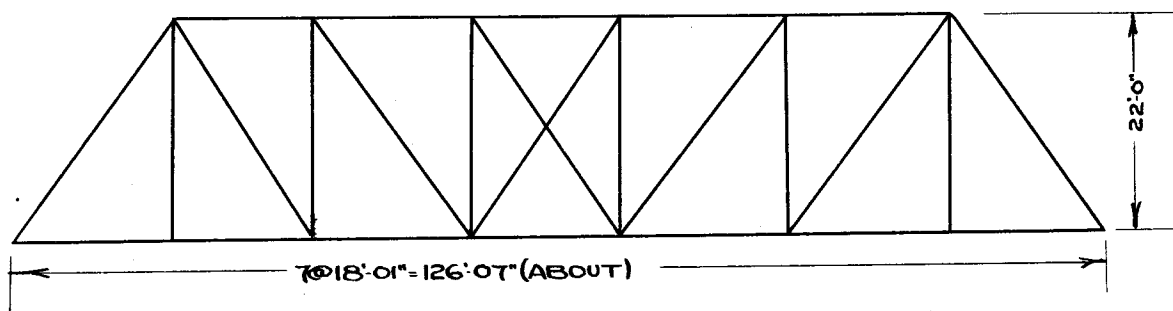
End shear on end floor beam is:

$$V = (2/3)37,800 + (2/3)11,340 + 45,500$$

$$V = 78,260\#$$

DESIGN OF TRUSSES

(Stresses in truss) The truss will be divided as shown below



Dead Load Stresses-

Wt. of concrete floor and wearing surface per. ft. of span is given by the formula

$$w^c = 124(T+1), T = \text{width of roadway}$$

$$= 124(39+1) = 4,960\# \text{ per. ft. of span.}$$

$$4,960/2 \cdot 18.08 = 44,800\# = \text{panel load per. truss due to dead load of concrete.}$$

Wt. of metal per. ft. of span is given by the formula

$$w = 4.5L + 460$$

$$= 4.5(128) + 460 = 1035\# \text{ per. ft. of span.}$$

$$1035/2 \cdot 18.08 = 9,350\# \text{ panel load due to metal weight.}$$

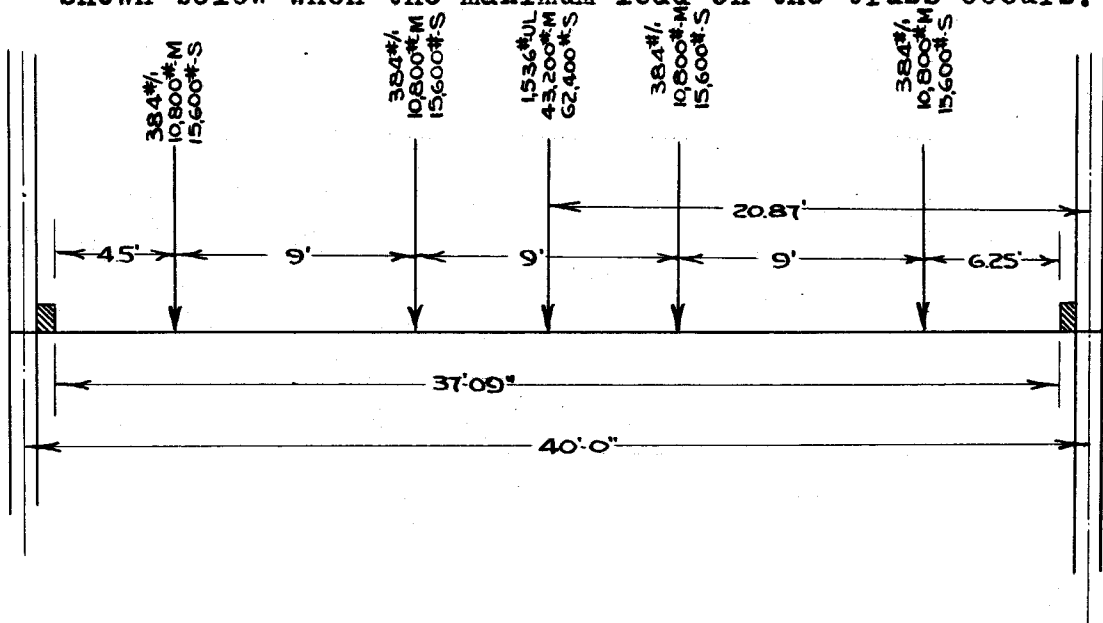
Total panel load due to dead load:

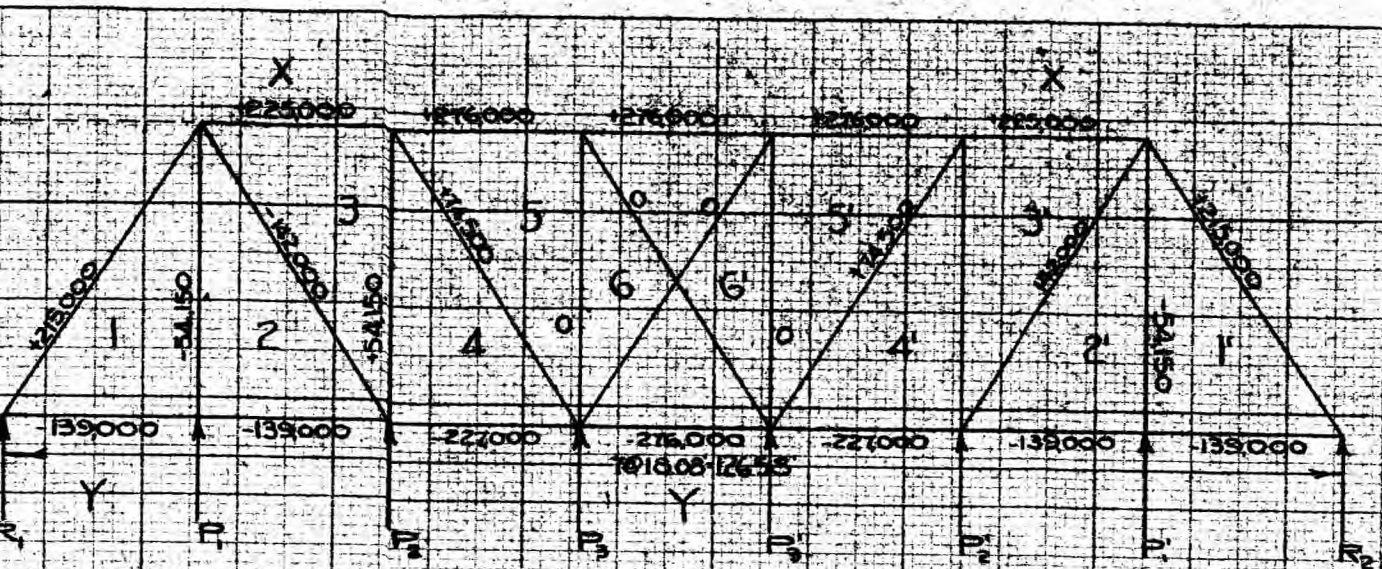
$$W = 44,800 + 9,350 = 54,150\#$$

Dead load stresses in the truss are determined by graphics on the print on the following page.

Live Load Stresses-

The distance c to c of trusses is assumed to be 40', curb 6", width of cover plate 15". Loading will be as shown below when the maximum load on the truss occurs.





JOINT LOAD $P = 54,150\#$

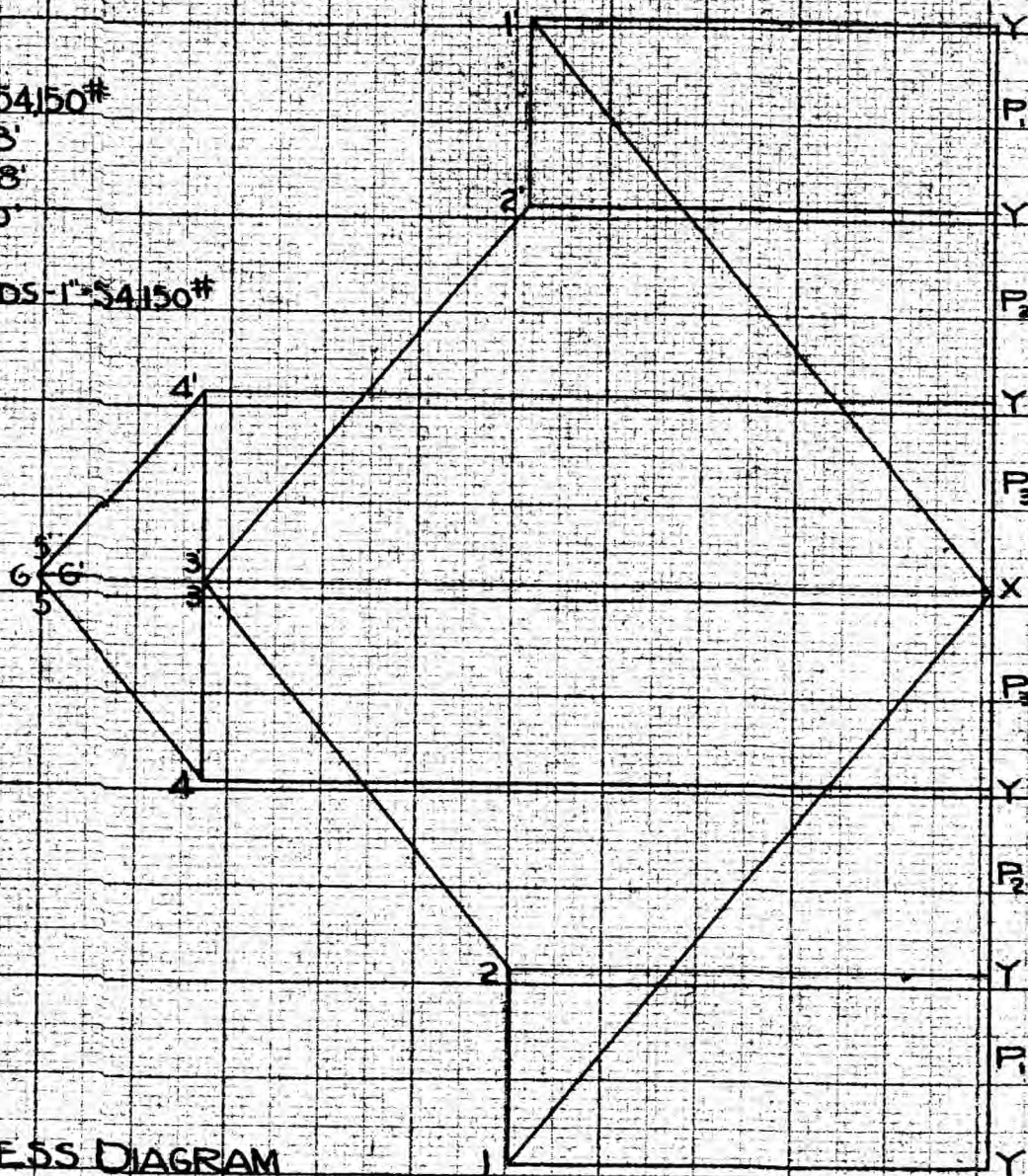
SPAN $L = 126.58'$

PANEL $L = 18.08'$

DEPTH $d = 22.0'$

$N = 7$

SCALE OF LOADS $1" = 54,150\#$



STRESS DIAGRAM
DEAD LOAD

Taking moments about R_2

$$R_1 = 1536(20.87)/40 = 800\#, \text{ which is the maximum uniform live load per. ft. truss.}$$

$$P_1 = (\text{Panel load due to uniform live load}) \\ = 800(18.08) = 14,500\#$$

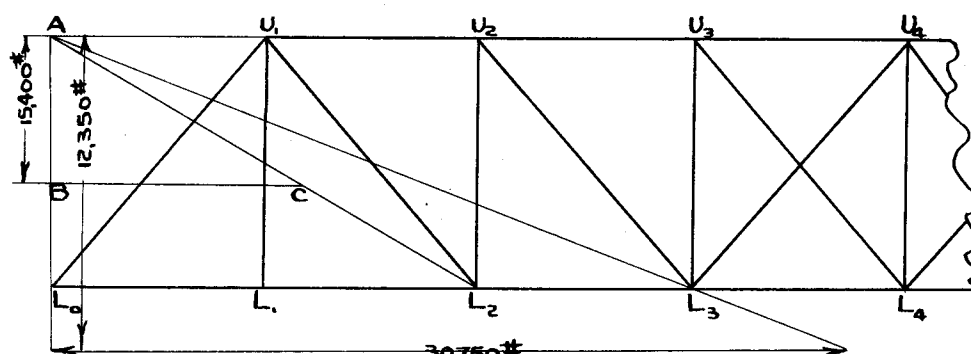
$$P_2 = (\text{Panel load due to concentration used for moment}) \\ = 43,200(20.87/40) = 21,600\#$$

$$P_3 = (\text{Panel load due to concentration used for shear}) \\ = 62,400(20.87/40) = 31,400\#$$

The stresses in the chords and end posts due to the uniform live load occurs when all panel points are loaded the same as dead load. These stresses are obtained by multiplying the dead load stresses by the ratio of the uniform live load to the dead load.

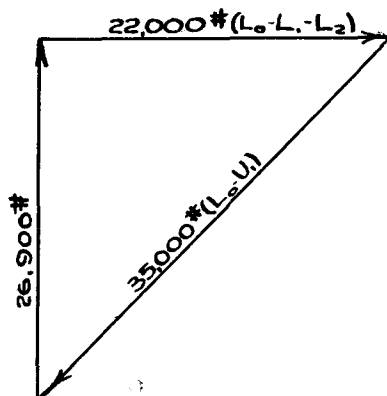
$$P'/W = 14,500/54,150 = .267$$

Multiplying this fraction by the chord stresses as already given, gives the chord stresses due to uniform live load. These stress are shown on the stress sheet.



The maximum stress in the end post L_0-U_1 occurs when the 31,400# concentration is at L_1 . The reaction at L_0 due to this concentration is $6/7 \cdot (31,400) = 26,900\#$. The maximum stress in the end post U_1-L_0 and the lower

chord $L_0-L_1-L_2$ is determined by graphics as shown below:



The maximum stress in the top chord U_1-U_2 due to the concentrated live load occurs when the 21,600# concentration is at L_2 . The reaction at L_0 is $(21600)5/7 = 15,400\#$. By graphics, as

shown on the preceding page, the stress in U_1-U_2 is 25,000#. Referring again to the same diagram, the maximum stress in the top chord U_2-U_3 due to concentration will occur when the 21,600# concentration is at L_3 . The reaction at L_0 is then $(21,600)4/7 = 12,350\#$. In the same manner as for determining the stress in U_1-U_2 , the stress in U_2-U_3 is determined. This same stress is the maximum stress in top chord U_3-U_4 and bottom chord L_2-L_3 .

The maximum live load stress will occur in post U_2-L_2 and diagonal U_2-L_3 when the 31,400# concentration is at L_3 and 14,500# of the uniform load is at each of the panel points L_3, L_4, L_5, L_6 . The reaction at L_0 under this loading is $(31,400)4/7 + (14,500)10/7 = 38,200\#$. This is the maximum stress in post U_2-L_2 . The stress in diagonal U_2-L_3 is $\sec \phi (38,200) = 1.55(38,200) = 59,200\#$.

The maximum live load stress in diagonal U_1-L_2 will

occur when the 31,400# concentration is at L_2 and the uniform 14,500# is at L_2, L_3, L_4, L_5, L_6 . The reaction at L_0 under this loading is

$$(31,400)5/7 + (14,500)15/7 = 53,500\#$$

$$53,500(1.55) = 82,900\#$$

The maximum live load stress in post U_3-L_3 and diagonal U_3-L_4 will occur when the 31,400# concentration is at L_4 and 14,500# of the uniform load is at each of the panel points L_4, L_5, L_6 . The reaction at L_0 under this loading is $(31,400)3/7 + (14,500)6/7 = 25,850\#$. This is the maximum live load stress in post U_3-L_3 . The stress in diagonal U_3-L_4 is $\sec \phi (25,850) = 1.55(25,850) = 40,000\#$.

Impacts-

$$L_0-L_1 = 50/(128+125) \cdot (57,400+35,000) = 18,050\#$$

$$U_1-U_2 = .19(60,000 + 25,000) = 16,800\#$$

$$U_2-U_3, U_3-U_4 = .19(73,600 + 30,750) = 20,600\#$$

$$L_0-L_1, L_1-L_2 = .19(37,000 + 22,000) = 11,650\#$$

$$L_2-L_3 = .19(60,500 + 30,750) = 18,050\#$$

$$L_3-L_4 = .19(73,600 + 30,750) = 20,600\#$$

$$U_1-L_1 = 50/(110+125) \cdot (45,900) = 9,770\#$$

$$U_1-L_2 = 50/(90+125) \cdot (82,900) = 19,876\#$$

$$U_2-L_2 = 50/(90+125) \cdot (38,200) = 8,880\#$$

$$U_2-L_3 = 50/(72+125) \cdot (59,200) = 15,000\#$$

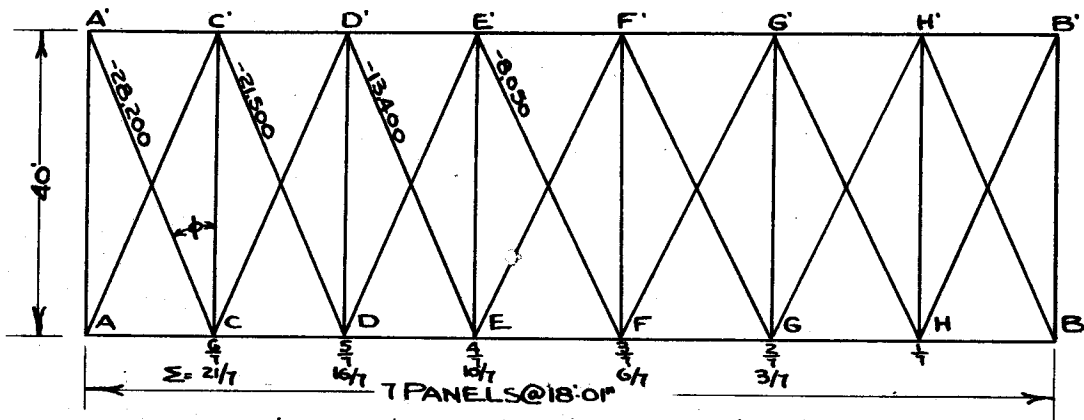
$$U_3-L_3 = 50/(54+125) \cdot (25,850) = 7,220\#$$

$$U_3-L_4 = 50/(54+125) \cdot (40,000) = 11,200\#$$

WIND STRESSES

(Stresses in Bottom Laterals) $1\ 1/2$ times the vertical projection of the lower half of the span is approximately 8 sq.

ft. per ft. of span. The live loadwind stress is $8(30) = 240\#$. The wind load is then $8(30) + 240 = 480\#$ per. ft. of bottom chord. The panel load is then $P = 480(18) = 8630\#$.



$$\tan \phi = 18/40 = .45 \text{ and } \sec \phi = 1.09$$

$$P \sec \phi = 8630(1.09) = 9,400\#$$

Referring to the above diagram, the stresses are as follows:

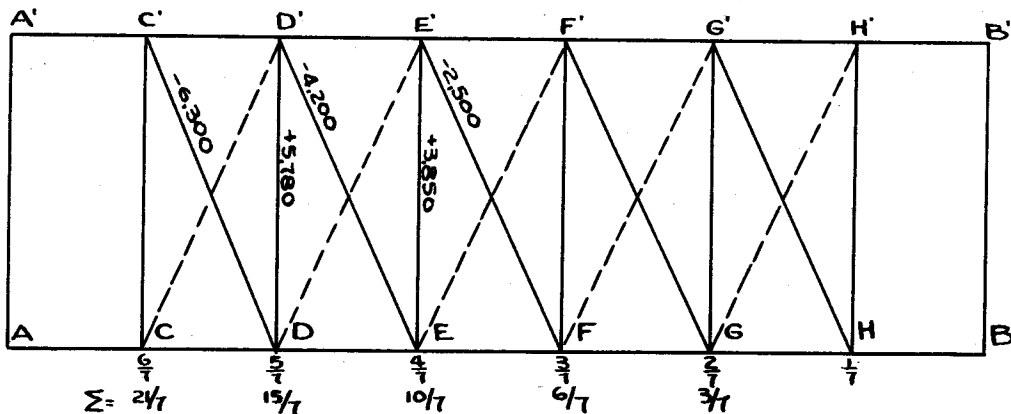
$$\text{Stress in } A'C = 9,400(21/7) = 28,200\#$$

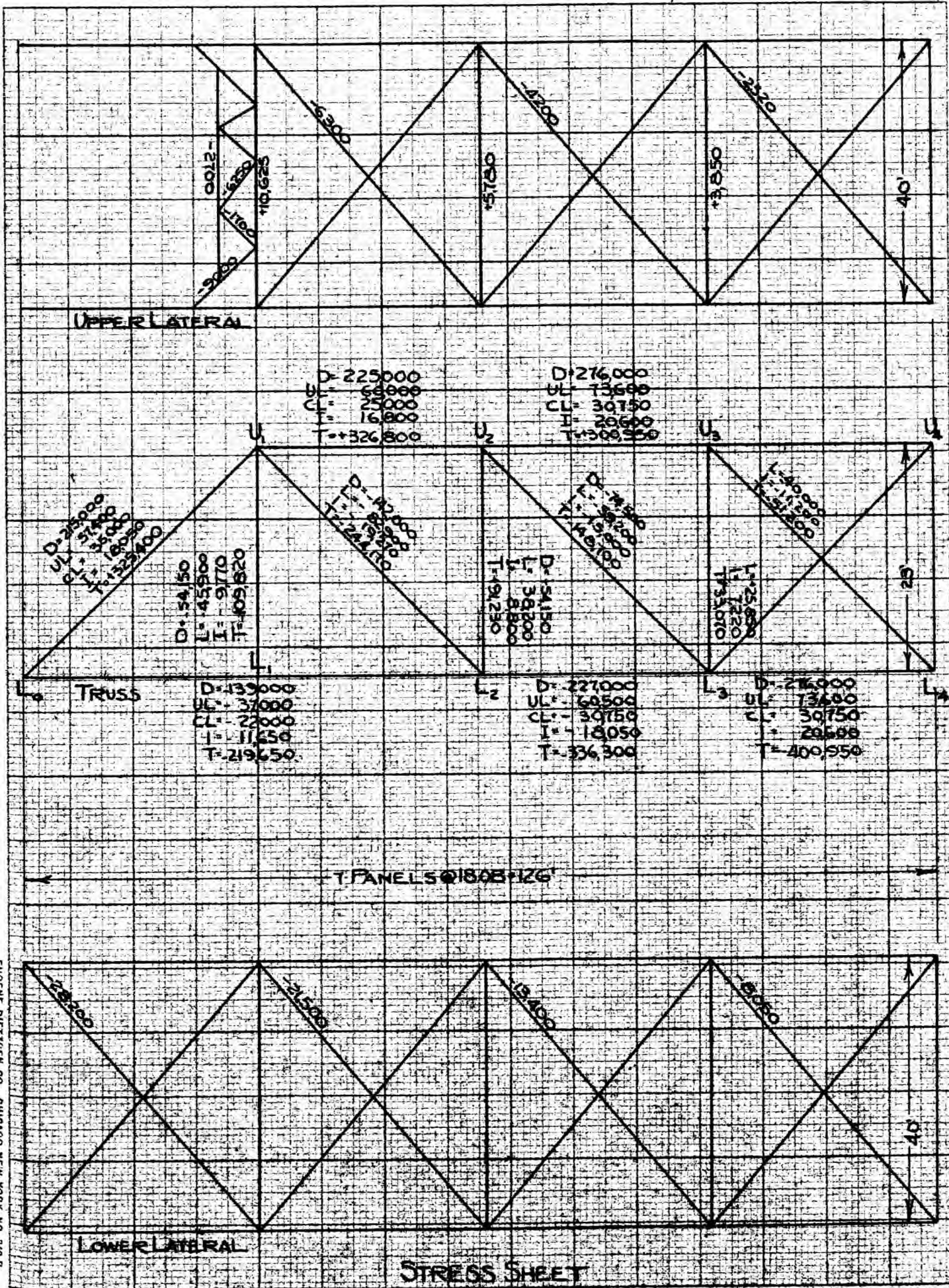
$$\text{Stress in } C'D = 9,400(16/7) = 21,500\#$$

$$\text{Stress in } D'E = 9,400(10/7) = 13,400\#$$

$$\text{Stress in } E'F = 9,400(6/7) = 8,050\#$$

(Stresses in top laterals) The wind stress will be considered as $150\#$ per ft. of truss, which is the minimum allowable. Then the panel load will be $150(18) = 2700\#(P)$ $\sec \phi = 1.09$ (same as for bottom laterals), and $P(\sec \phi) = 1.09(2700) = 2,940\#$.





Referring to the diagram on the preceeding page:

$$\text{Stress in } DC' = 2,940(15/7) = 6,300\#$$

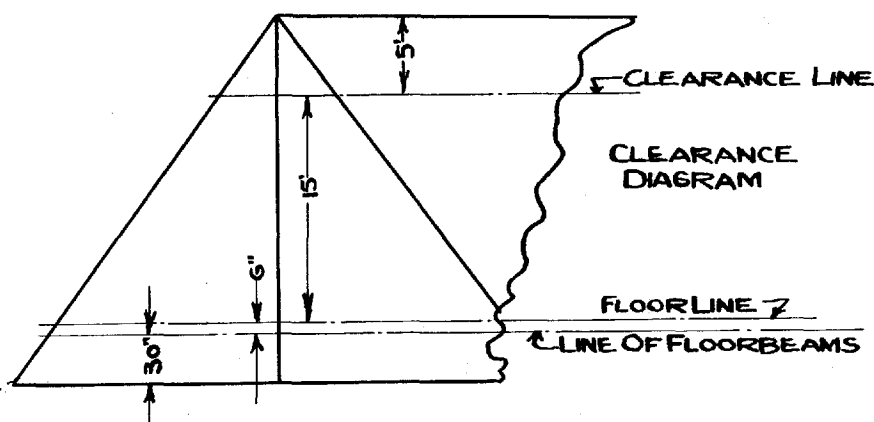
$$\text{" " } ED' = 2,940(10/7) = 4,200\#$$

$$\text{" " } FE' = 2,940(6/7) = 2,520\#$$

$$\text{" " } \text{strut } DD' = 2,700(15/7) = 5,780\#$$

$$\text{" " " } EE' = 2,700(10/7) = 3,850\#$$

(Wind Stresses in Portals) There can be two panel loads applied to the portal at either side, which load is $2,700(2) = 5,400\#$. End posts are considered fixed, the point of contraflexure being midway between the bottom of the portal and the shoe joint L_0 .



From the clearance diagram above it is found that the portal can be 8' deep. The total length of the end post is about 29'. The distance from the shoe joint to the bottom of the portal is $29 - 8 = 21'$, and the point of contraflexure is $21/2 = 10.5'$. Each end post has equal resistance to horizontal shear, which is $2700\#$ at point of contraflexure. The $5400\#$ load will cause a positive reaction at one point of contraflexure and a negative moment at the other. The reaction at O, or O' (see print on next page) is $5400(18.5)/21 = 4750\#$. Stresses are now found by graphics, drawing substitute frame O-P-Q.

DESIGN OF SECTIONS

(Intermediate Post)- Wide flange beams will be used for this section. Depth of truss is 23'. A Beth. G-12 at 61.0# gives 2.31 as the least radius of gyration. $L/r = 23(12)/2.31 = 120$, which is the limit for slender ratio. From curve, 7,750# is the allowable unit stress. $91,230/7,750 = 11.78$ sq. in. area required. The beam has an area of 17.92 sq. in., which is excessive, but will be used as it proves to be most economical. This section will also be used for U_3-L_3 , proving to be most economical.

(Hanger U_1-L_1)- The required area is $109,820/18,000 = 6.1$ sq. in. The same section as used for U_2-L_2 will be used, giving an effective area of $17.92 - 4(.38) = 16.40$ sq. in., which is excessive but will be used. (G-12 at 61.0#)

(Diagonals U_1-L_2)- The required area is $244,170/18,000 = 13.6$ sq. in. The same section as used for U_1-L_1 will be used here, giving an effective area of 16.40 sq. in. (G-12 at 61.0#)

(Diagonal U_2-L_3)- The required area is $148,700/18,000 = 8.25$ sq. in. A Beth. B-12 at 36# gives an effective area of $10.58 - 4(.38) = 9.06$ sq. in. and will be used.

(Diagonal U_3-L_4)- The required area is $51,200/18,000 = 2.85$ sq. in. Two angles $3\frac{1}{2}$ by $2\frac{1}{2}$ by $5/16$ give an effective area of $2(1.78) - 2(.27) = 3.05$ sq. in., and will be used.

(Bottom Chord $L_0-L_1-L_2$)- $219,650/18,000 = 12.2$ sq. in. for the required area. 4 angles, 6 by 4 by $7/16$ give an effective area of $4(4.18) - 2(2)(7/8)(7/8) = 13.71$ sq. in., and will be used.

(Bottom Chord L_2-L_3)- $336,300/18,000 = 18.7$ sq. in. for the re-

quired area. 4 angles, 6 by 4 by 9/16 give an effective area of $4(5.31) - 2(7/8)(18/16) = 19.71$ sq. in. and will be used. (Bottom Chord L_3-L_4)- $400,950/18,000 = 22.3$ sq. in for the required area. 4 angles, 6 by 4 by 11/16 give an effective area of $4(6.40) - 2(7/8)(22/16) = 23.20$ sq. in. and will be used for this section.

(Top Chord U_1-U_2)- 12,000# will be used as the allowable unit stress. $326,800/12,000 = 27.2$ sq. in. for the required area of the cross section. The following section will be assumed:

One cover plate 20" by 1/2"

Two channels 15" at 35#

Total section area = 30.58 sq. in.

The least radius of gyration for this built up section is about the horizontal axis and is 5.84. The value of $L/r = 18(12)/5.84 = 37$. For this value, the allowable unit stress is $16,000 - 70(37) = 13,410\#$. The required area is therefore $326,800/13,410 = 24.4$ sq. in. The section chosen is slightly in excess of this figure but will be used.

(Top Chord $U_2-U_3-U_4$)- The same section as for U_1-U_2 will be used as that section will prove satisfactory.

(End Post U_1-L_0)- The same section as chosen for the top chords will be assumed, giving an area of 30.58 sq. in., and a least radius of gyration of 5.84. The end post is 29' long, so the value of $L/r = 29(12)/5.84 = 59.5$ (about). The allowable stress is therefore $16,000 - 70(59.5) = 11,800\#$ per sq. in.(app.). The required section area is $325,450/11,800 = 27.6$ sq. in. The chosen section is 30.58 sq. in. and will be used.

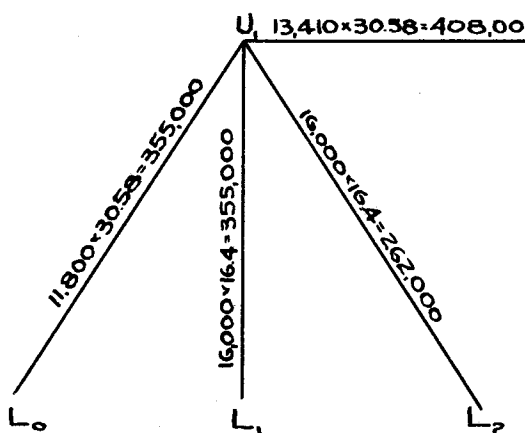
(Bottom Laterals)- The maximum stress in the end bottom laterals is 28,200#. The required area is $28,200/18,000 = 1.56$ sq. in. One angle, 3 by 3 by 5/16 gives an effective area of $1.78 - (5/16)(7/8) = 1.55$ sq. in. and will be used. All other laterals in the bottom will be made of this section.

(Top Laterals)- The maximum stress is -6,300#. One angle, 4 by 3 by 5/16 will be used.

DESIGN OF JOINTS

All joints will be designed to develop the full strength of the member. All gusset plates will be made 1/2" thick. Rivets shall be 3/4" in diameter. The gage line of angles will be placed on the center line of the truss. When an angle has two gage lines, the one nearest the back is used. The center of gravity of the top chord and end post are placed on the center line of the truss.

(Joint U_1)- The gusset plate will be shop riveted to U_1-U_2 and field riveted to each of the other members. Bearing con-



trols the number of rivets in

L_0-U_1 , and the number of field rivets required will be

$355,000 / (.75)(.37)20,000 = 64$, or

32 on each side. The number of

shop rivets for U_1-U_2 is

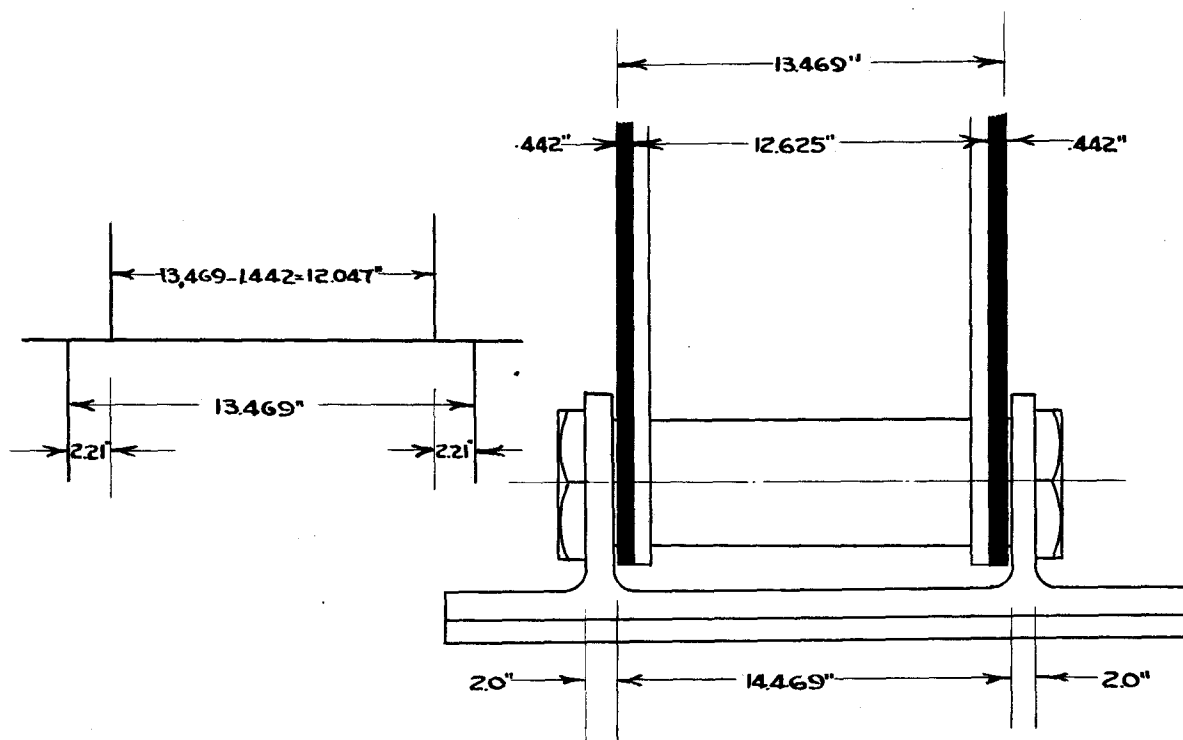
$408,000 / (.75)(.37)24,000 = 62$, or

31 on each side. The number of field rivets in U_1-L_2 is de-

termined by single shear and is $262,000/4,420 = 60$, or 30 ri-

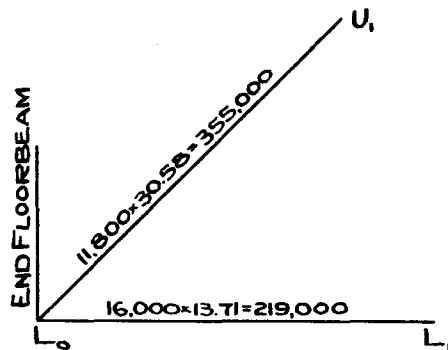
vets on each side. The number of field rivets in hanger U_1-L_2

is determined by single shear and is the same as for U_1-L_2 . (Joint L_0)- Cast iron shoes will be used at the fixed end, while cast iron rockers will be used at the expansion end. A 4" pin will be used if it proves satisfactory. The forces acting on the pin are all vertical and equal to one-half the sum of the dead load, live load, impact, times the number of panels. This equals $(1/2)(44,800+31,400)7 = 133,350$. The minimum thickness of the gusset plate $t = 133,350/4(24000) = 1.39"$. The thickness of the web is $.422"$, so the required thickness of the plates is $1.39 - .422 = 0.97"$. A 1" gusset plate will be used.



The maximum bending moment is $2.211(66,675) = 147,200$ in. lbs. The maximum shear is 66,675#. The diameter of the pin required by moment is $d = \sqrt[3]{10.2M/S_b} = \sqrt[3]{10.2(147,000)/24,000} = 3.95"$. The diameter required by shear is $d^2 = 1.27(66,675)/S$

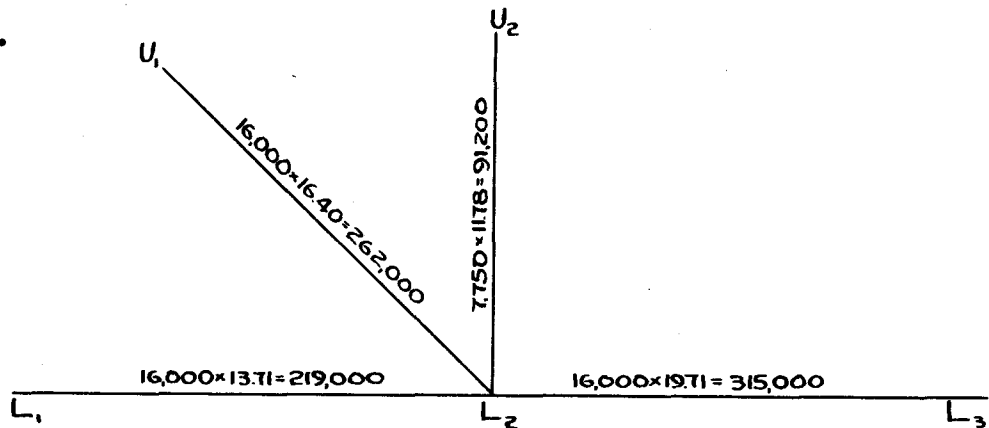
$d^2 = 1.27(66,675)/12,000 = 7.08$ and $d = 2.59$. The 4" pin is satisfactory.



Rivets required in $L_0-U_1 = 355,000 / (.42)(.75)(24,000) = 48$, or 24 on each side. (shop).

Rivets required in $L_0-L_1 = 219,000 / 4,420 = 50$, or 25 on each side. (field)

(Joint L_2) - The gusset plate will be shop riveted to L_2-L_3 at L_2 .



Rivets required in $U_1-L_2 = 262,000 / 4,420 = 60$, 30 on each side.

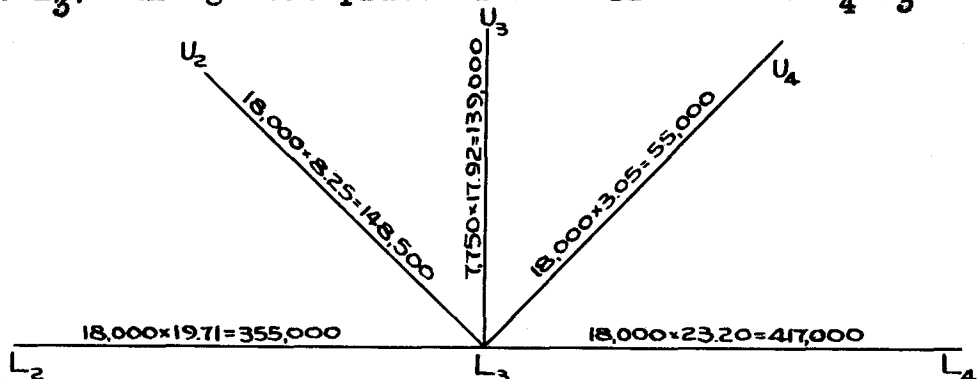
" " " $L_1-L_2 = 219,000 / 4,420 = 50$, 25 " " " .

" " " $L_2-L_3 = 315,000 / 5,300 = 60$, 30 " " " .

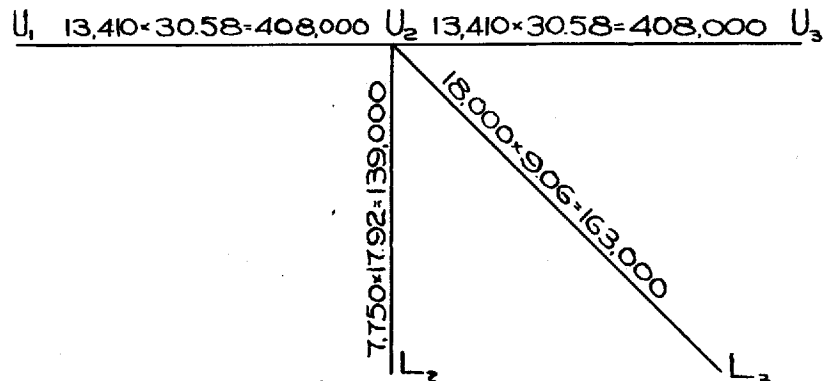
" " " $U_2-L_2 = 91,200 / (.5)(.75)(16,000) = 16$, 8 on

each side. All of the above are field rivets except those for L_2-L_3 , which are shop rivets.

(Joint L_3) - The gusset plate will be riveted to L_4-L_3 at L_3 .



Rivets for $L_2-L_3 = 355,000/4,420 = 80$, 40 on each side. (field)
 " " $L_4-L_3 = 417,000/5,300 = 88$, 44 " " " (shop)
 " " $U_2-L_3 = 148,500/4,420 = 34$, 17 " " " (field)
 " " $U_3-L_3 = 139,000/6,750 = 20$, 10 " " " (")
 " " $U_4-L_3 = 55,000/4,420 = 12$, 6 " " " (")
 (Joint U_2) - Gusset plate shop riveted to U_1-U_2 at U_2 .



Rivets for $U_1-U_2 = 408,000/9,000 = 46$, 23 on each side. (shop)
 " " $U_3-U_2 = 408,000/7,500 = 54$, 27 " " " (field)
 " " $L_2-U_2 = 139,000/7,500 = 18$, 9 " " " (")
 " " $U_3-U_2 = 148,700/4,420 = 34$, 17 " " " (")

END PLATE DESIGN

The maximum pedestal reaction is 133,350# and the area of each masonry plate must be $133,350/600 = 221$ sq. in. (600# per. sq. in. allowable bearing stress on concrete masonry). A plate 12" by 2" by 24" will be used giving an area of 288 sq. in. The length of the rocker arm will be taken as 24", so the bearing stress between the rocker and the plate is $133,350/24 = 5,550$ # per. lin. in. The allowable is 600d (used d = 18", diameter of rocker) = $600(18) = 12,800$ # lin. in.

COST OF PRATT TRUSS BRIDGE

The weight estimate of the Pratt truss bridge is given in the table below.

The table below.							
No. pcs.	Shape	Section	Length Ft. in.		Wt. per. ft. lb.	Weight each	Total
Upper Lateral System							
6	I-Beams	B-15	27	8	35	1,339	7,980
10	Angles	4x3x5/16	41	4	7.2	295	2,952
Portal System							
8	Channels	15"	27	8	35	965	7,800
4	Cover Pl.	20"x1/2	26	7	34	901	3,604
4	Angles	4x3x5/16	13	6	7.2	102	408
4	"	"	22	0	7.2	154	637
4	"	"	6	5	7.2	46	185
4	"	"	38	6	7.2	276	1,704
4	"	4x3x1/2"	6	5	11.2	72	288
Truss Members							
8	Channels	15"	36	5	35	1,278	10,224
2	Cover Pl.	20x1/2"	92	0	34	3,128	6,256
4	Channels	15"	54	10	35	925	7,700
2	Posts	G-12	20	11	61	1,220	2,440
2	"	"	20	11	61	1,281	2,562
2	Laterals	"	21	0	61	1,285	2,570
2	"	"	25	11	61	1,586	3,172
2	Beam	B-12	26	0	36	936	936
4	Angles	3x2x5/16	25	2	1.78	45	180
16	"	6x4x7/16	53	7	14.3	772	12,352
8	"	6x4x11/16	18	1	21.8	383	3,064

No. pcs.	Shape	Section	Length ft. In.	Wt. per. ft. lb.	Weight each	Total
Hand Railing						
4	Angles	2 1/2 x 2 x 1/4	128 0	3.6	461	1,844
4	"	3 x 3 x 1/2"	4 3	49.5	40	160

Floor System

6	Beams	BG-30	38 10	220	8,580	51,480
2	"	"	"	190	7,410	15,820
91	Stringer	B-12	18 0	28	504	<u>45,864</u>

Total Wt. 191,082

Floor Slab Concrete = $(6.5/12)(38)(128)/27 = 97$ cu. yd.

Gravity piers will be used as in the plate girder.

FINAL COST OF BRIDGE

191,082# structural steel in place at \$0.05	\$9,554
Details, 5%, 9,555# structural steel in place at \$0.05=	477
97 cu. yd. concrete (floor slab) at \$20.00	1,940
273 sq. yd. wearing surface at \$0.60	164
2 gravity piers at \$1,200.00	<u>2,400</u>
Total cost -	\$14,535

CONCLUSION

A table of the cost of the four bridges is given below:

Cost of Reinforced Concrete Bridge	- \$16,979
Cost of Encased Steel Type Bridge	- 19,316
Cost of Plate Girder Bridge	- 19,796
Cost of Pratt Truss Type Bridge	- 14,535

From the above table it is seen that from the standpoint of first cost only, the Pratt bridge is some \$2,000.00 the more economical. It must be remembered that no type selection is made here and many things are to be taken into consideration before a selection would be made. Due to the 40 ft. width of roadway used, the weights of members in the steel bridges are very large and for this reason the cost of these bridges may be somewhat in excess since a flat rate of \$0.05 per. lb. for structural steel in place was used.

The margin of cost difference for the concrete structures is due, no doubt, to the great difference in span lengths, the encased steel type span being double that of the reinforced.

DESIGN TABLES

**Table Number I. Areas and Summation of Perimeters
also Weights of Bars**
Summation of Areas = A. Summation of Perimeters = 0

Bar Size	Wt. Lbs.	Number of Bars							
			1	2	3	4	5	6	7
3/8"0	0.376	A	0.11	0.22	0.33	0.44	0.55	0.66	0.77
		0	1.18	2.36	3.53	4.71	5.89	7.07	8.25
1/2"0	0.668	A	0.20	0.39	0.59	0.79	0.98	1.18	1.38
		0	1.57	3.14	4.71	6.28	7.85	9.42	10.99
1/2"S	0.850	A	0.25	0.50	0.75	1.00	1.25	1.50	1.75
		0	2.00	4.00	6.00	8.00	10.00	12.00	14.00
5/8"0	1.043	A	0.31	0.61	0.92	1.23	1.53	1.84	2.15
		0	1.96	3.93	5.89	7.85	9.82	11.79	13.74
3/4"0	1.502	A	0.44	0.88	1.33	1.77	2.21	2.65	3.09
		0	2.36	4.71	7.07	9.42	11.78	14.14	16.49
7/8"0	2.044	A	0.60	1.20	1.80	2.41	3.01	3.61	4.21
		0	2.75	5.50	8.25	11.00	13.74	16.49	19.24
1" 0	2.670	A	0.79	1.57	2.36	3.14	3.93	4.71	5.50
		0	3.14	6.38	9.42	12.57	15.71	18.85	21.99
1" S	3.400	A	1.00	2.00	3.00	4.00	5.00	6.00	7.00
		0	4.00	8.00	12.00	16.00	20.00	24.00	28.00
1 1/8S	4.303	A	1.27	2.53	3.80	5.06	6.33	7.59	8.86
		0	4.50	9.00	13.50	18.00	22.50	27.00	31.50
1 1/4S	5.313	A	1.56	3.12	4.69	6.25	7.81	9.38	10.94
		0	5.00	10.00	15.00	20.00	25.00	30.00	35.00

Areas and Summations of Perimeters for Various Spacings.

Bar		Spacing of Bars in Inches							
Size		3"	4"	5"	6"	7"	8"	10"	12"
3/8"0	A	0.44	0.33	0.26	0.22	0.19	0.17	0.13	0.11
	0	4.71	3.53	2.33	2.36	2.04	1.73	1.41	1.18
1/2"0	A	0.78	0.59	0.47	0.39	0.34	0.29	0.26	0.20
	0	6.28	4.71	3.77	3.14	2.69	2.36	1.88	1.57
1/2"S	A	1.00	0.75	0.60	0.50	0.43	0.37	0.30	0.25
	0	8.00	6.00	4.80	4.00	3.43	3.00	2.40	2.00
5/8"0	A	1.23	0.92	0.74	0.61	0.53	0.46	0.37	0.31
	0	7.85	5.89	4.71	3.93	3.36	2.94	2.35	1.96
3/4"0	A	1.77	1.32	1.06	0.88	0.76	0.66	0.53	0.44
	0	9.42	7.06	5.65	4.71	4.04	3.53	2.83	2.36
7/8"0	A	2.40	1.80	1.44	1.20	1.03	0.90	0.72	0.60
	0	10.00	8.24	6.59	5.50	4.71	4.12	3.30	2.75
1" 0	A	3.14	2.36	1.88	1.57	1.33	1.18	0.94	0.78
	0	12.57	9.42	7.54	6.28	5.26	4.71	3.77	3.14
1" S	A	4.00	3.00	2.40	2.00	1.71	1.50	1.20	1.00
	0	16.00	12.00	9.60	8.00	6.86	6.00	4.80	4.00
1 1/8S	A	5.06	3.80	3.04	2.53	2.17	1.89	1.52	1.27
	0	18.00	13.50	10.80	9.00	7.72	6.85	5.40	4.50
1 1/4S	A	6.25	4.69	3.75	3.12	2.68	2.34	1.87	1.56
	0	20.00	15.00	12.00	10.00	8.57	7.50	6.00	5.00

TABLE No. II
Values of k, j, p, and K

n	f _c	f _s = 16000				f _s = 18000			
		k	j	p	K	k	j	p	K
10	1200	0.428	0.857	0.0161	220.4	0.400	0.867	0.0133	208.3
	1250	0.438	0.854	0.0171	234.0	0.410	0.863	0.0142	221.3
	1300	0.448	0.851	0.0182	247.8	0.419	0.860	0.0157	234.5
	1350	0.457	0.848	0.0193	261.8	0.428	0.857	0.0161	247.8
	1400	0.466	0.845	0.0204	276.0	0.437	0.854	0.0170	261.4
12	850	0.389	0.870	0.0103	143.2	0.361	0.880	0.0085	134.3
	900	0.402	0.866	0.0113	150.0	0.375	0.875	0.0093	146.8
	950	0.415	0.863	0.0123	170.0	0.388	0.871	0.0102	150.0
	1000	0.422	0.857	0.0134	183.5	0.400	0.867	0.0111	172.3
	1050	0.441	0.853	0.0145	197.3	0.412	0.863	0.0120	186.3
	1100	0.453	0.849	0.0156	211.5	0.423	0.859	0.0129	199.8
	1150	0.464	0.845	0.0167	225.8	0.434	0.855	0.0138	212.7
	1200	0.474	0.842	0.0178	239.8	0.444	0.852	0.0148	226.3
15	600	0.358	0.881	0.0067	94.4	0.333	0.889	0.0056	88.9
	650	0.379	0.874	0.0077	107.5	0.351	0.883	0.0063	100.8
	700	0.396	0.868	0.0087	120.4	0.368	0.877	0.0072	113.1
	750	0.413	0.862	0.0097	133.5	0.385	0.872	0.0080	125.7
	800	0.429	0.857	0.0107	146.9	0.400	0.867	0.0089	138.7
	850	0.444	0.852	0.0118	160.9	0.415	0.862	0.0098	151.9
	900	0.458	0.848	0.0129	174.5	0.429	0.857	0.0107	165.4

f_s = 20000

n	f _c	k	j	p	K
10	1200	0.375	0.875	0.0113	197.0
	1250	0.384	0.872	0.0120	209.2
	1300	0.392	0.869	0.0128	222.5
	1350	0.403	0.866	0.0136	235.1
	1400	0.412	0.863	0.0144	248.9
12	850	0.338	0.897	0.0070	127.4
	900	0.351	0.883	0.0079	139.4
	950	0.363	0.879	0.0086	151.6
	1000	0.375	0.875	0.0094	164.1
	1050	0.386	0.871	0.0101	176.2
	1100	0.399	0.867	0.0109	189.7
	1150	0.408	0.864	0.0118	202.6
15	1200	0.419	0.860	0.0126	216.1
	600	0.310	0.897	0.0047	83.5
	650	0.323	0.891	0.0053	94.9
	700	0.344	0.885	0.0060	106.6
	750	0.369	0.880	0.0068	118.6
	800	0.375	0.875	0.0075	131.2
	850	0.389	0.861	0.0083	144.0
	900	0.403	0.861	0.0091	157.0

Based on the Formulae.

$$n = \frac{E_s}{E_c}$$

$$k = \frac{1}{1 + \frac{f_s}{nf_c}}$$

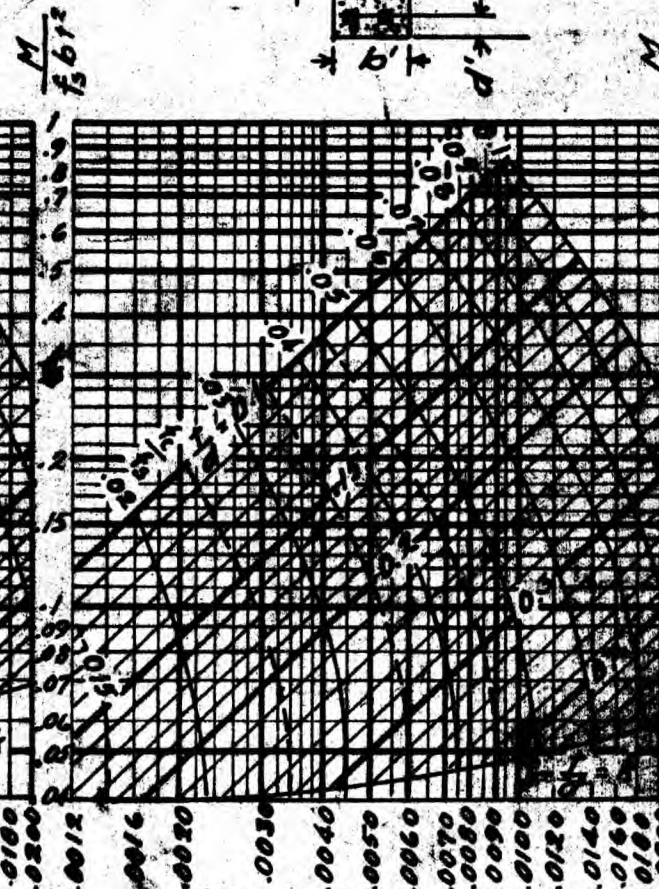
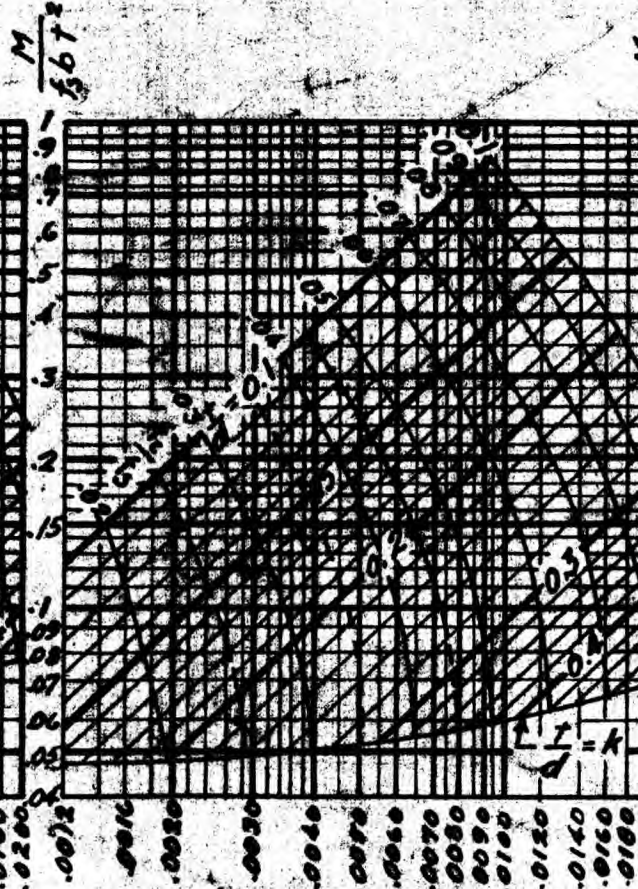
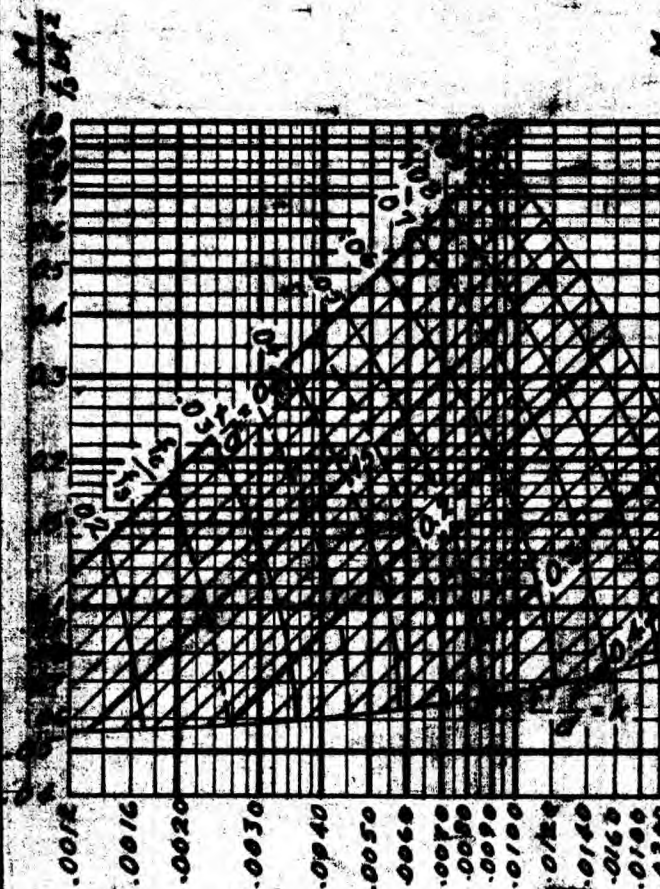
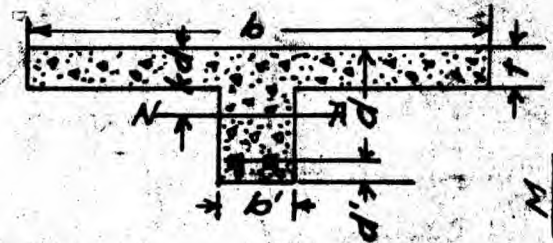
$$p = \frac{f_{ck}}{2f_s}$$

$$j = \frac{1}{1 + \frac{f_s}{f_c} \left(\frac{f_s}{nf_c} + 1 \right)}$$

$$j = 1 - \frac{k}{3}$$

$$K = f_s p j = \frac{1}{3} f_{ck} j$$

"T" BEAMS Neutral Axis in Web.



$$N = 10$$

Formulae. $\frac{t}{d} = \sqrt{R_2^2 - 12 \frac{f_c}{f_s} R_1}$

$$p = \frac{t}{d} \frac{f_c}{f_s} - \left(\frac{t}{d} \right)^2 \frac{1}{2} \left(\frac{f_c}{f_s} + \frac{1}{n} \right)$$

$$N = 12$$

$$R_1 = \frac{f_c}{f_s} + \frac{1}{n}$$

$$R_2 = 3 \left(\frac{M}{f_s b t^2} + \frac{f_c}{f_s} + \frac{1}{2n} \right)$$

$$N = 15$$

STIRRUP SPACING

$V_c = v_b j d$ = Shear Carried by Concrete
 V_s = Shear Carried by Stirrups
 S = Stirrup Spacing in Inches
 f_s = Shearing Stress in Steel
 A_s = Area of both stirrup bars

Stirrups must be hooked

Formulae :

$$S = \frac{A_s f_s j d}{V_s}$$

Max. Spacing

$$S = \frac{45}{\alpha + 10} d = 0.45d$$

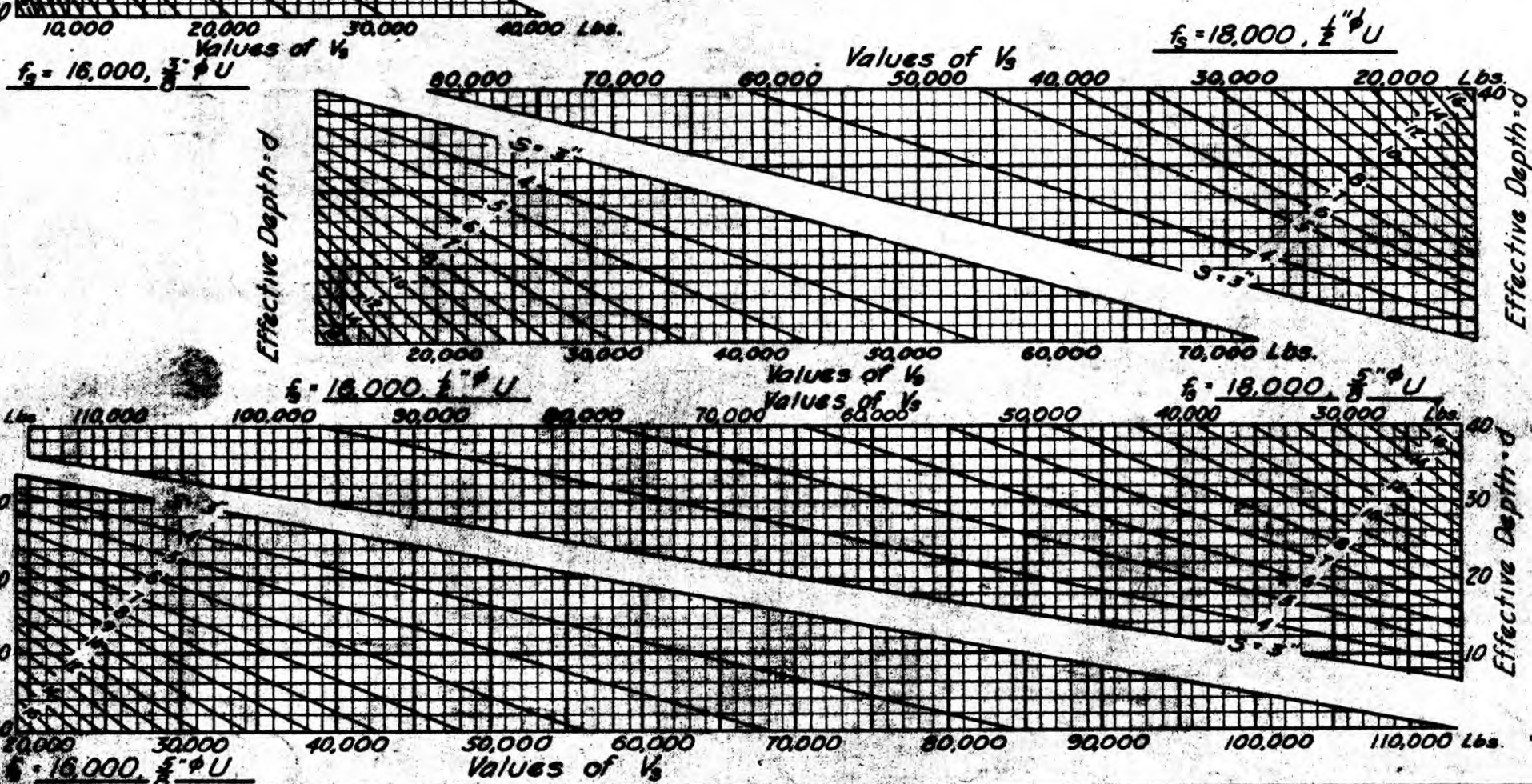
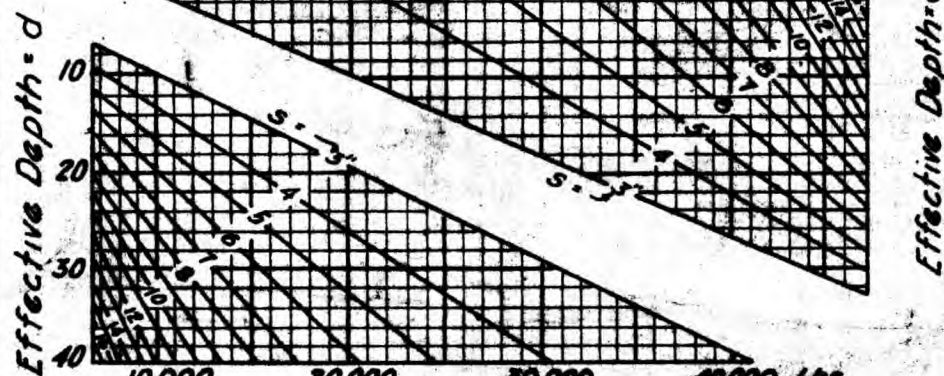


DIAGRAM No. 3

RECTANGULAR ECCENTRIC COLUMNS Compression over the Entire Section.

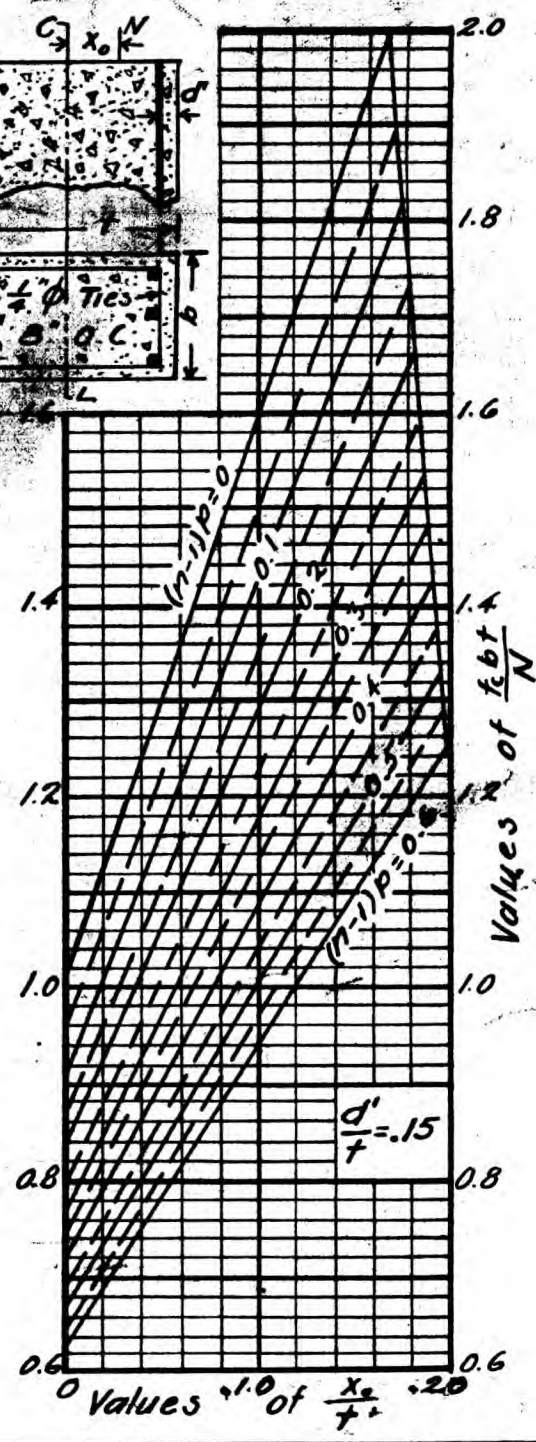
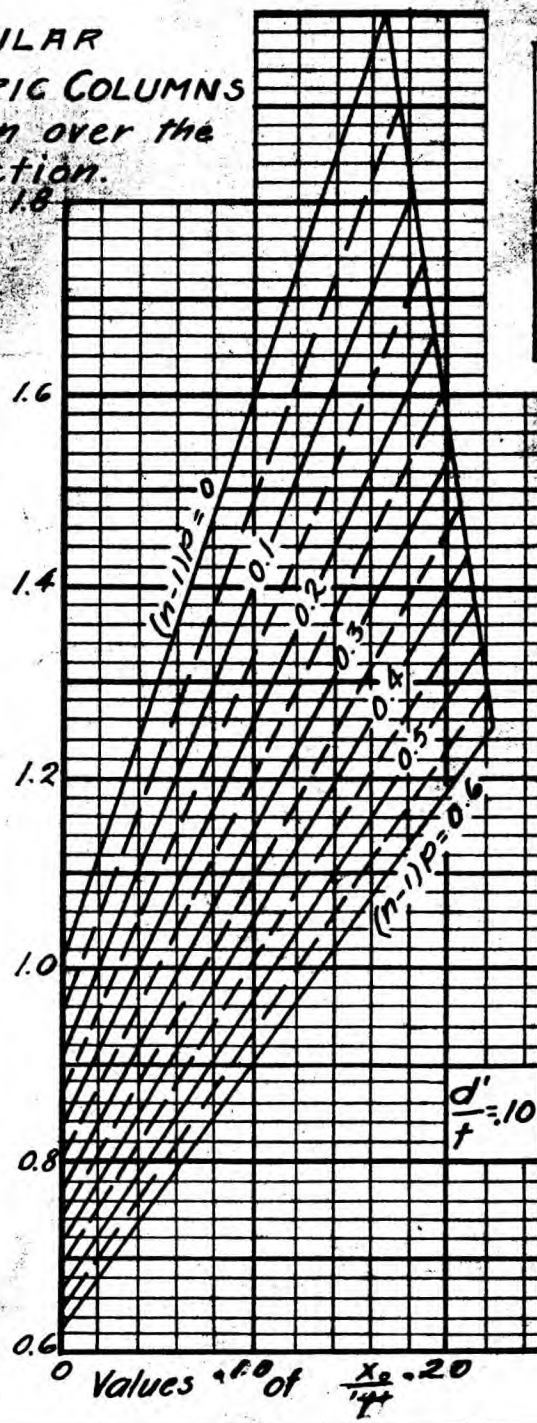
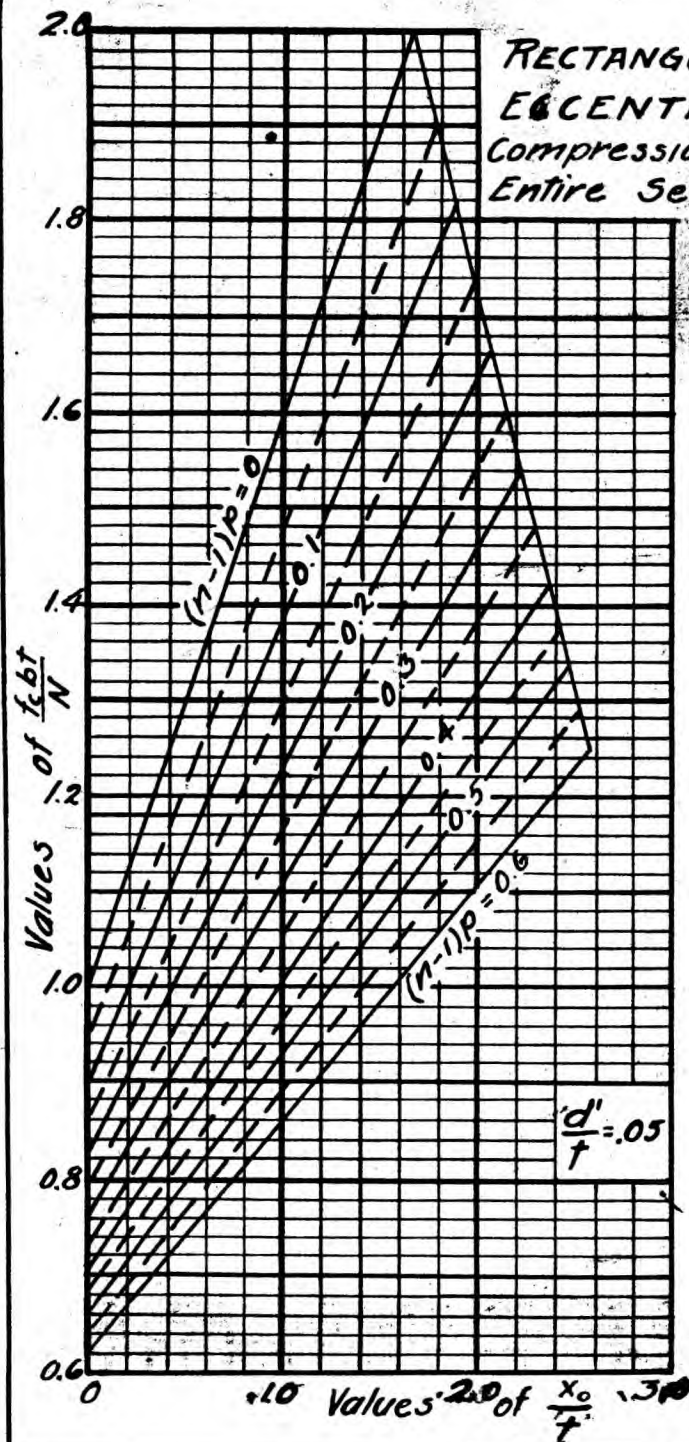


DIAGRAM 4

RECTANGULAR ECCENTRIC COLUMNS Tension over Part of the Section.

